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STATE-OF-THE-ART IN PREDICTING PAVEMENT RELIABILITY FROM INPUT VARIABILITY

W. Ronald Hudson

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for

U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
P. O. Box 631, Vicksburg, Miss. 39180

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16. Abstract

This report describes and discusses the concept of reliability in pavement design. Some designers refer rather to "design confidence level" or even in an older sense to increased design safety factors. The report covers the improvement of pavement design and the fact that "pavement management systems" methodologies provide an ideal framework for considering reliability.

Consideration is given to three techniques for reducing variability in design: (1) improved quality control, (2) use of design confidence level, and (3) the concept of reliability level. The study shows that pioneers in realistic use of reliability in design include Kher, Lemer, Moavenzadeh and Darter.

The philosophy of determining and corroborating reliability levels is discussed and recommendations for work to advance the state-of-the-art are presented.

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A

PREFACE

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The help of Drs. B. F. McCullough, T. W. Kennedy, and Messrs. Harvey Treybig and Harold von Quintus in providing information for and review of this manuscript is gratefully acknowledged.

METRIC CONVERSION FACTORS

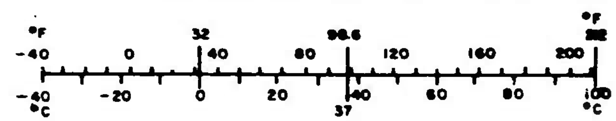
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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INTRODUCTION

The purpose of this report is to discuss the consideration of variability and reliability in pavement design. A good deal is known about variability of pavements and input variables. That subject has been discussed in detail by Kennedy, Hudson, and McCullough (Ref 47) and will not be repeated here. In this report will be discussed concepts of reliability and how they relate to pavement design model reliability, materials variability, load variability, and actual pavement performance.

The nature of practically all of the factors involved in the pavement design system described above is stochastic (probabilistic). Due to lack of knowledge and information and uncertain future social-economic conditions, many design factors cannot be exactly predicted; and there are also inherent along-the-roadway variations in pavement strength due to nonhomogeneous materials and variable construction practices. This uncertainty in prediction and natural variations of important parameters results in variations in pavement system performance and in some pavement sections "failing" before others. This variable nature of failure or distress may be observed along every in-service pavement. Essentially, this uncertainty results in a certain amount of early failure before the "average" or desired life has occurred. The analysis of these types of variabilities and uncertainties may be handled through a so-called probabilistic or stochastic approach although most existing design procedures do not account for them effectively.

In structural and foundation design, the various uncertainties have been provided for by empirical safety factors. This generally has resulted in few failures, but has probably resulted many times in an overdesign and sometimes in underdesign of the structure, depending on the magnitude of variations and the level of applied safety factors. The use of arbitrary safety factors in pavement design is normally questioned because human lives are not generally endangered when a pavement wears out or fails the way they are if a building or bridge fails.

The minimization of costs while satisfying the performance requirements is the objective of pavement design. However this must be done in the face of variability. Using the probabilistic approach, it is often possible to quantify the design risk and to design for a specified level of reliability. The

exact definition of this specified reliability is a more difficult problem.

In this report we will try to present the background of pavement design and performance concepts in order to establish a framework for considering variability and in setting allowable levels of reliability. Then several methods in current use for considering one or more aspects of variability in existing design methods are reviewed as are some concepts for evaluating variance of several kinds affecting the problem. Finally the state-of-the-art is summarized and recommendations for subsequent action are presented.

Background

The purpose of a pavement is to carry load and to provide some adequate level of service to its user. The load may be small as in the case of a sidewalk or bicycle path; or large as in the case of an Air Force bomber base pavement. The required level of service will also vary widely depending on vehicle speed and traffic volume. Prior to 1958 little was done to relate pavement service to the user's needs except implicitly by the designer. At the beginning of the AASHO Road Test there was no definable pavement performance concept. This resulted in difficulty in defining pavement test section failure at the WASHO Road Test (Ref 27). According to W. N. Carey, Jr., Chief Engineer at WASHO and later Chief Engineer on the AASHO Road Test, the expert pavement engineers of the early fifties could not easily reach agreement on when a pavement test section had failed at the WASHO Test. In order to conduct the AASHO Road Test, it was necessary to fill this void in the understanding of pavement performance. As a result, the pavement serviceability-performance concept was developed (Ref 3) by Paul Irick and W. N. Carey, Jr. This so-called "PSI" concept permits the definition of the current level of service provided by the pavement, e.g. its present serviceability (PSI). The PSI is defined as the ability of a pavement to serve high-speed, high-volume traffic, e.g. the user, safely and conveniently. The accumulated serviceability history of the pavement is termed its "performance." A specified level of serviceability can be selected as the minimum acceptable in a particular case of pavement use. This value is often taken as 2.5 on the scale of 5.0 for interstate highways and as 2.0 or 1.5 for secondary roads (Fig 1).

No similar system of defining pavement "failure" exists for airfield pavements. Existing FAA and U.S. Department of Defense design procedures use rather arbitrary definitions of pavement failure such as (1) the development of

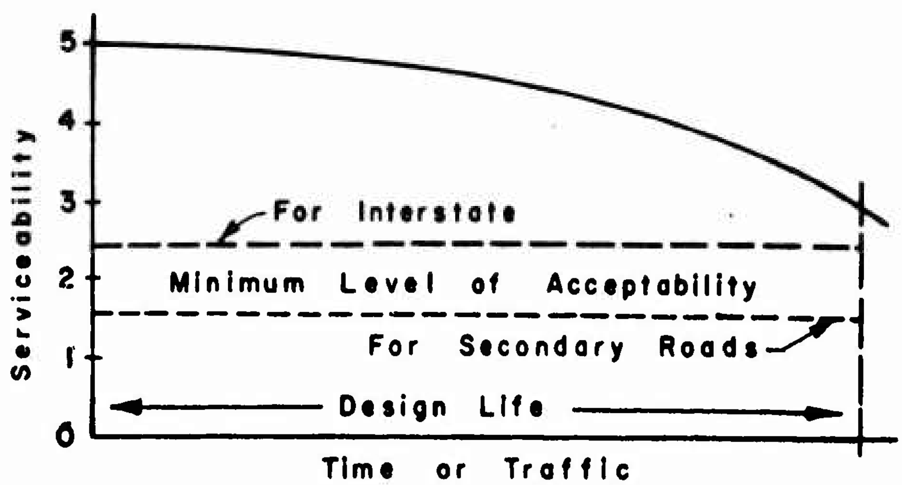


Fig 1 The serviceability-history - performance concept illustrated with possible minimum levels of acceptability shown.

first crack, (2) the breaking of a concrete slab into six pieces, (3) the development of 1.5 inches of pavement deformation, or others.

These are recognized as arbitrary statements of failure and in point of fact, the decision of "failure" is always a subjective one, both before and after the pavement is designed and built. In order to apply reliability concepts to airfield pavement design, it will be essential that an objective method of defining pavement failure be established. Preliminary work on this subject has been done by McCullough and Stietle (Ref 46). More detail is presented on this subject later herein.

Design and the System Concept

Historically, pavement "design" has been thought of as a single phase process which could be done with complete confidence if we could only develop "rational" methods. Gradually engineers are becomeing aware that this is an unrealistic definition of design. In reality "design" is a part of a continuing process of providing adequate pavement in the face of variability and uncertainty. Even in complex design and manufacturing problems, the concept of quality control and setting confidence limits on the product are becoming well accepted. As a result of the serviceability-performance concept, the life cycle of a pavement can be shown in terms of a serviceability-time or traffic history curve where the pavement is constructed at a given level of serviceability and reaches the minimum acceptable level of serviceability at the end of its "design life." (Fig 1)

Thus any pavement which will carry the expected traffic for the design period would presumably be an "acceptable design." On the other hand, any pavement which falls below the minimum acceptable level of serviceability prior to the end of the design life or analysis period would be unacceptable. Such a simplistic analysis cannot realistically consider pavement maintenance nor rehabilitation. This has been handled in the past by assuming that "normal maintenance" would be performed as necessary on any acceptable design.

In 1967-68 Hudson, Finn, et al (Ref 7) extended this concept to show the pavement performance curve as a system output function for a pavement system (Fig 2). We will not attempt here to discuss systems design or pavement systems in detail. Those are well covered by others (Refs 6, 7, 8, 9, 25). However, it should be pointed out that the understanding of the pavement as a system and the application of system reliability concepts to pavement management in all probability hold the key to the proper consideration of

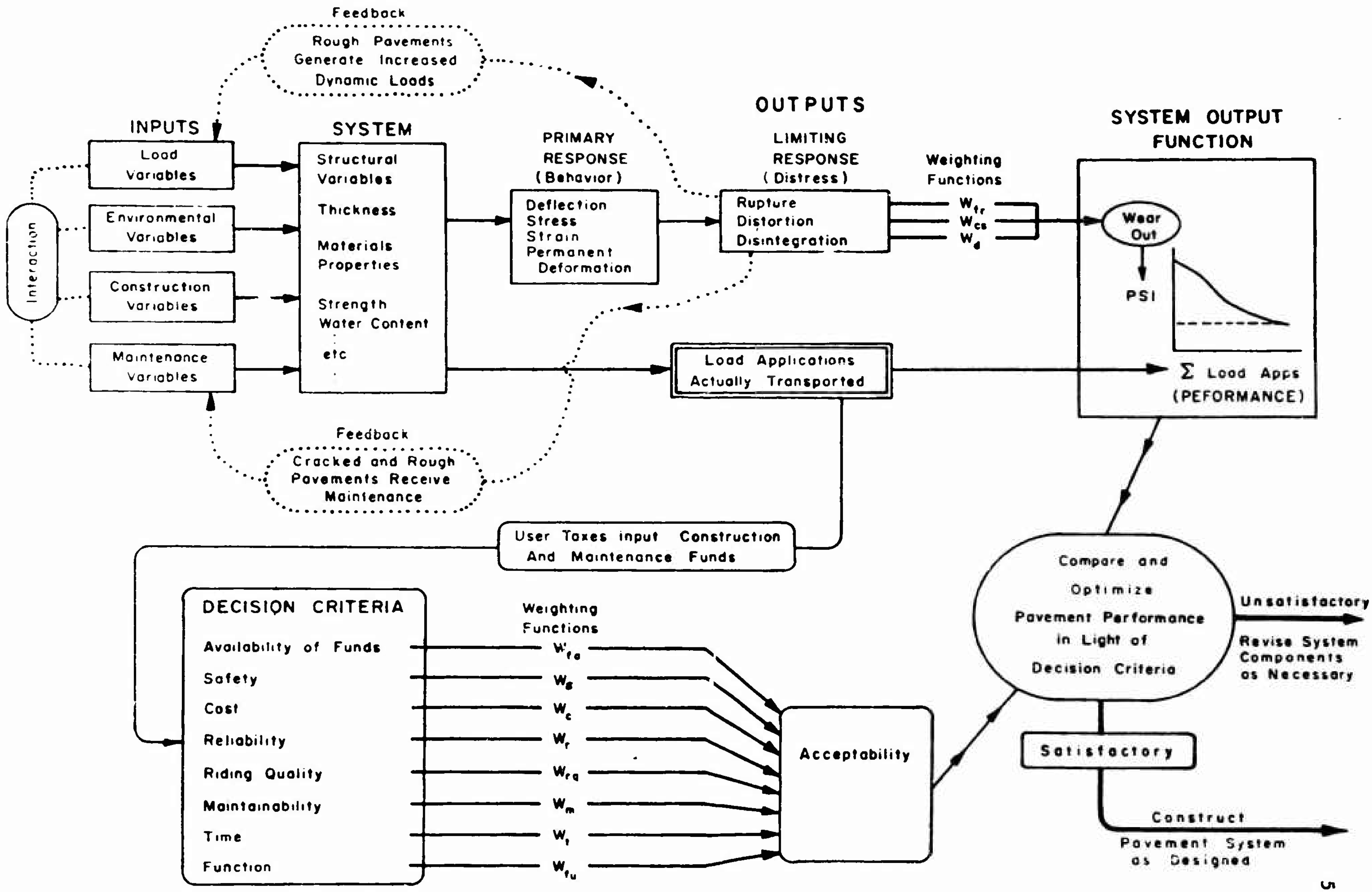


Fig 2 Conceptual pavement system (after NCHRP1-10, Ref 20)

reliability in pavement design and performance.

The systems concept recognized maintenance as an important part of providing adequate pavement life and showed both construction and maintenance variables as input data to the system. In 1970 a team of researchers from the Texas Cooperative Highway Research Program presented a working pavement design concept which carries the rational pavement design one step further (Ref 9). It not only included maintenance as an important and necessary input variable, but the concept recognized that most pavements are not in fact constructed to remain smooth for the entire design life without additional work (Fig 3), but rather that most pavement designs involve two or more performance periods in which (1) a pavement is constructed at an initial serviceability level, (2) it deteriorates to an unacceptable level, (3) it is repaired or rehabilitated, and (4) it continues to serve traffic.

This process may be repeated several times during the life of the pavement depending upon the desires of the designer and the acceptance of the user. These desires are quantified in terms of constraints and cost factors in the pavement system.

In September, 1970, Haas and Hutchinson (Ref 6) coined the phrase "management system" with reference to highway pavements. As they outlined it, the pavement must be (1) designed, (2) the design communicated for implementation, (3) constructed, (4) maintained, (5) monitored for feedback information, and (6) rehabilitated as needed one or more times for the total design life (considering cost and all required inputs).

Thorough examination of actual highway pavement life histories by several state highway departments as well as on airfields by the U.S. Air Force and the U.S. Corps of Engineers indicate that this cyclic process is much more realistic than the so-called one-shot design method. As a matter of fact, almost no pavements can be found that serve out a predetermined design life of twenty years or more without some major maintenance or rehabilitation. At the recent Highway Research Board pavement systems Workshop held in Austin, Texas (Ref 17) these concepts were thoroughly discussed and generally accepted by a wide variety of pavement design and research engineers.

Changes in Input Variables

The concept becomes even more realistic when we realize that a change in any of the key input variables from the estimates in the original design

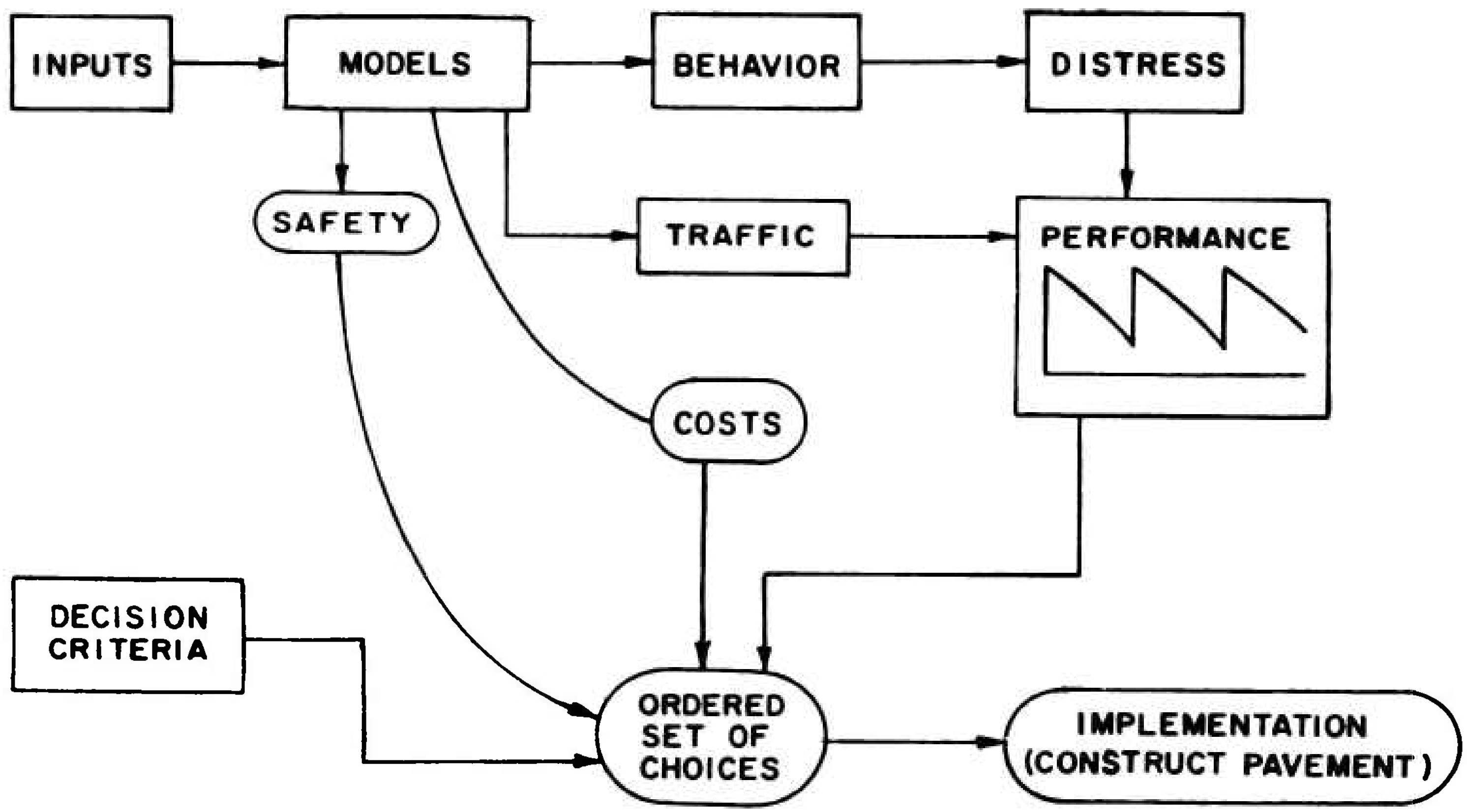


Fig 3 Simplified block diagram of the pavement system (after Ref 25)

problem can result in a significantly altered performance history for the pavement. For example, if the traffic on a highway is significantly increased over the estimate, due to the development of a new industrial park, the estimated pavement life can be shortened markedly. Likewise if the environmental conditions are considerably better than predicted, the pavement may last longer than expected. If worse moisture conditions prevail, a shorter life could result, etc.

Optimization of Pavement Design

Building on the concept outlined above, the pavement management concept consists of several cycles in which the pavement system is repeatedly analyzed and examined. The first of these might be called the design cycle. In general terms, subsequent cycles could meaningfully be called rehabilitation cycles. The initial or design cycle of the process then involves the selection of the optimum performance history for the initial structural section based on the input data and imposed constraints of the problem as illustrated in Fig. 4. As a part of this process, some initial set of materials and geometry is selected for construction.

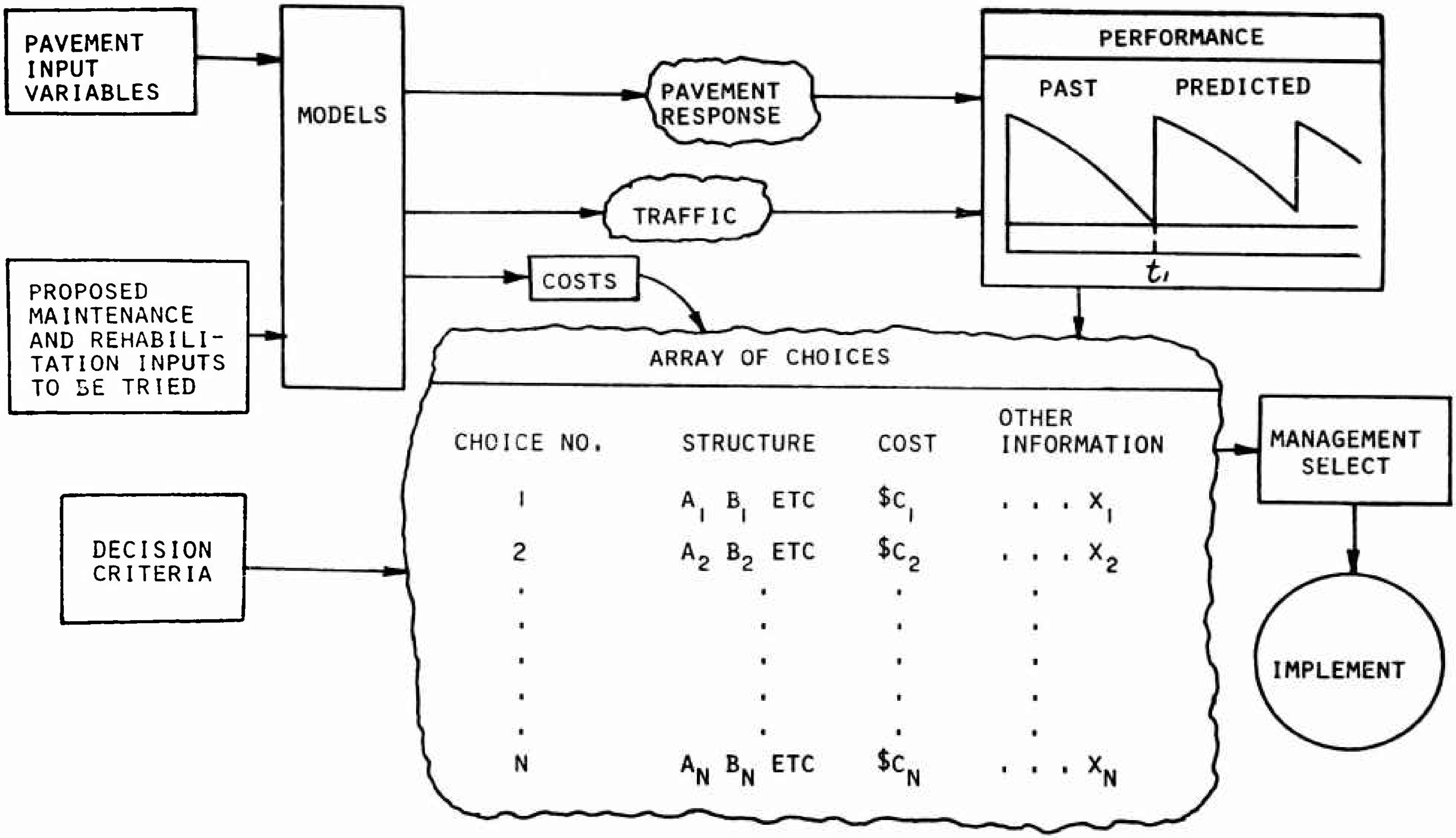


Fig 4 Conceptual pavement system structured to illustrate the array of choices and costs

HISTORICAL CONSIDERATION OF VARIABILITY

Prior to 1965 nearly all pavement design concepts were basically deterministic in formulation if not in fact. The design of portland cement concrete pavements is useful in illustrating the historical perspective. In the accepted methods an equation relating stress to load was used as the basic design model. All these have the same general form derived by Westergaard (Ref 19).

$$\sigma_c = \frac{3.0P}{h^2} \left[1 - \left(\frac{a_1}{\ell}\right)^{0.6} \right] \qquad (1)$$

σ_c = maximum tensile stress in pounds per square inch at the top of the slab, in a direction parallel to the bisector of the corner angle and at a distance of $2 \; a_1 \ell$ from the corner;

μ = Poisson's ratio for concrete;

E = modulus of elasticity of the concrete in pounds per square inch;

k = subgrade modulus in pounds per square inch;

a_1 = $2 \; a$ where a is radius of area of load contact in inches;

ℓ = radius of relative stiffness, defined by

$$\sqrt[4]{\frac{ED^3}{12(1 - \mu^2)k}} \qquad (2)$$

P = wheel load in pounds

h = slab thickness in inches

In this formula and the others like it, it was necessary for the designer to enter a "design" value for each of the parameters in order to predict the resulting stress. As outlined by Kennedy and Hudson (Ref 47) the chances of predicting all these parameters correctly for any given instance is equal to the area under a single point on the probability density function, that is, zero.

Also inherent in the method is the variability of load placement which actually ranges widely from the assumed "corner" position.

The remaining task for the designer was to select a level of concrete strength in his design. Experience had shown that it was not satisfactory to use the average concrete strength of a sample mix design because premature failure would result. This led the designer to modify his design to provide a so-called factor of safety against failure.

Factor of Safety Designs

Early testing of composite materials such as concrete and asphalt concrete seemed to indicate the existence of a fatigue limit for these materials of approximately 50% of ultimate strength. That is to say that laboratory tests seemed to indicate that the material would last indefinitely if subjected to repeated stresses not exceeding 50% of the ultimate strength. Subsequently it has been shown that these projections were due to inadequate laboratory data due to the exceptionally long testing times required for load repetitions in excess of 1×10^6 (Ref 43). This concept was taken as a "theoretical" basis for using a safety factor of 2.0 in pavement design. That is, the slab thickness was selected so that:

$$\sigma_c \leq S_c/2.0 \tag{3}$$

where

σ_c = calculated load corner stress,

S_c = ultimate concrete strength, and

2.0 = the safety factor

Temperature Stresses

Ignored in the calculations of these designs were the stresses due to factors other than traffic loads, including shrinkage, warping, and other

temperature effects. In fact, Westergaard (Ref 26) showed that these stresses can often exceed load stresses. Likewise, Abou-Ayyash et al (Ref 48) have calculated that environmental stresses alone can be sufficiently large to cause cracking in the slab under certain conditions.

By cut and try over several years, it was determined that this safety factor procedure was approximately adequate for pavements on well-drained subgrades. But, in fact the safety factor could better be called an "ignorance factor" since it provides adequate strength to cope with variability and with temperature stresses rather than setting a real fatigue limit. Furthermore, the fact that most loads do not travel exactly at the edge or corner added to the "safety."

First Crack vs. Failure Index

To compound the problem, the methods as typified above can only predict the first crack in the pavement slab even if they are perfect models, and first crack does not constitute failure of the pavement in the sense of carrying traffic. Thus, the stress calculated really served only as an indicator or "index" of the total stress condition history and it is foolish to think of any of the safety factor methods as being theoretically well-founded.

Furthermore, this type of pavement design concept is not broad enough to serve as the basis for understanding and quantifying design reliability. Thus, it will not be considered further herein.

This does not imply that a factor of safety is not a valid concept for some other uses, but merely that when applied globally as outlined above, it cannot provide an adequate framework for handling reliability and variability.

Flexible Pavement Design Development

While the example above relates to rigid pavements similar processes were taking place in flexible pavement design methods. Methods such as the Combined CBR and the Texas Triaxial were developed and continually modified by experience to provide "adequate designs." Many methods made use of elastic layer theory to estimate load stresses and thus define required thicknesses of "better material" or provide a reasonable definition of "load equivalency." The methods were adjusted as design curves were "moved" to accommodate new "data" from more pavements in the field, in an attempt to

provide an "envelope" below all possible failures. These adjusted design curves constituted use of a more subtle "safety factor" embedded in the charts.

Statistical Quality Control

During the 1940's and 1950's there was considerable "premature" failure of pavements designed by safety factor methods and empirical methods. This led to a renewed search for additional "reliability" in pavement design methods. The procedures were modified, tried, and modified again, but little progress was made in stating design reliability of pavements in quantitative terms for improved design methods.

In the post-war years, considerable progress was made in the quality control field in manufacturing. In the mid-1950's these concepts were applied to highway construction. The materials work at the AASHO Road Test (Ref 28) began to show the benefits and limitations of applying quality control procedures to pavement materials. Excellent work was done by many state highway departments in this field (Refs 29, 30, 31, 32, 33, 34).

An excellent summary of the quality assurance field (as termed by the FHWA) is given in a series of articles headed by McMahon and Halsted. These concepts have been very useful in improving the uniformity of pavement materials and provide a basis for stating one half the problem of reliability.

Application to Design

There is immediate applicability of quality assurance to improved design as outlined by McCullough, Hudson, et al (Ref 7). The concept can best be illustrated by extending our Westergaard design example. The AASHO Interim Design Guides provided an early basis for looking at pavement failure as the summation of pavement distress and roughness. Thus, McCullough points out the value of predicting with statistical reliability the cracking history of a pavement. McCullough takes the Westergaard type equation discussed above as an indicator of pavement stress. In the Westergaard analysis, two classes of variables (load variables and structural variables) are considered directly. The structural variables are represented by the modulus of elasticity of the concrete, Poisson's ratio, thickness of the pavement, and modulus of subgrade reaction. The load variables are represented by the wheel load.

Cracking is considered to be a deterministic phenomenon occurring when the concrete stress is greater than the flexural strength, or a fatigue phenomenon occurring in a predictable relationship between repetitions of load and magnitude of load.

To predict the amount of cracking in the pavement, statistical methods must be utilized. Thus, the mean value and the standard deviation for the expected stress and also for the strength in the concrete pavement must be determined. The working stress determined from Westergaard's equation could have a mean value and a distribution if the variation of the material properties are known and used in the computations for stress; i.e., modulus of elasticity and Poisson's ratio. To determine the distribution of concrete strength, the flexural test data of the concrete may be processed to obtain a mean and a standard deviation. Using these values in a statistical analysis, the probability of a crack in a given area may be computed.

Figure 5 demonstrates that the probability of distress increases with an increase in variability of material properties. The solid lines in the figure represent normal distribution for induced stress, and the dash lines for flexural strength. For both cases, the mean induced stress is assumed to be 500 psi, and the mean strength is 600 psi which, in a purely deterministic analysis, should be adequate to prevent cracking. With a standard deviation of 20 psi for both flexural strength and induced stress, the area of overlap of the two probability functions is 1.2 percent. This area is related to the probability of cracking as is discussed below. If the standard deviation for the strength is increased to 40 psi (as may result from poor quality control on the job, for example), the area of overlap increases to approximately 9 percent.

In other words, as the variability of the flexural strength increases, the likelihood increases that the strength will be sufficiently smaller than the average for failure to occur. Failure actually occurs when strength is less than stress; thus, as is illustrated in Fig. 5, there is generally a greater failure probability if there is a large overlap area between the probability functions for strength and stress.

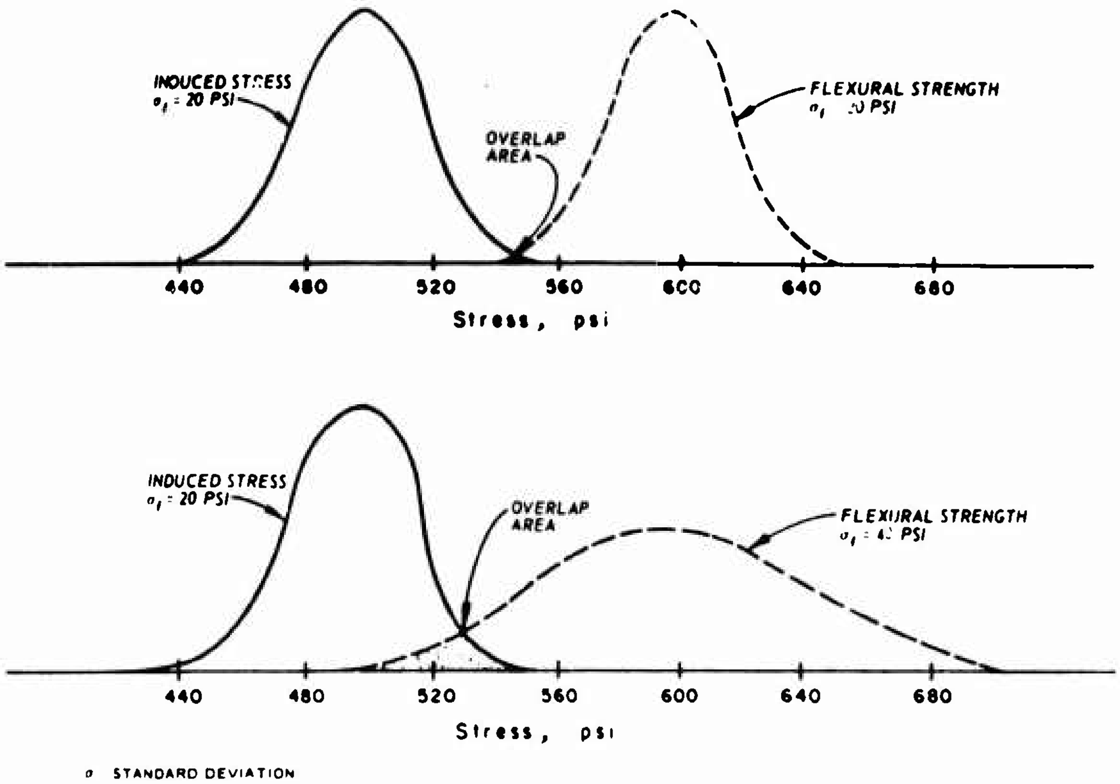


Fig. 5 Graphical presentation of failure for two conditions. (after McCullough, Ref 7)

The mathematical calculation of the failure probability is developed in the following chapter.

Using this approach, the expected area of cracking or distress may be computed for the condition. For example, using the 9 percent overlay area obtained above let us assume that for these conditions, there is a 3.5 percent probability of cracking. In a given random trial of applying a load stress to samples with these strength characteristics, there is a 3.5 percent probability that the specimen will fail due to overstress. Thus, it is reasonable to assume that in a very large number of trials, approximately 3.5 percent of the specimens would fail. Likewise considering a given area of roadway to be made up of a very large number of specimens, it could be hypothesized that approximately 3.5 percent of the roadway area would experience cracking under the load stress pattern outlined.

Model types similar to the model presented here for computing cracking due to the distress mechanism of excessive load are also required for the various other distress mechanisms active in the pavement. If all these models were available, the designer might predict the history of a distress index during the life of a roadway and thus he could predict the drop in the serviceability index with number of load applications. Or, he could use the area of intersection of the two curves (so-called failure area) as an "Index of Reliability."

There is no widespread use of this type of reliability concept. It is used in slightly different form by Treybig et al (Refs 36, 37) to provide a simple statement of reliability and to provide a quantitative way for the designer to vary his design by use of confidence levels.

<u>Statistical Confidence Levels</u>

One approach to considering variability in design is the concept of statistical confidence levels. Basically this concept involves adjusting design to consider parameters at some level other than the mean or average value. In some ways this resembles the old safety factor concept, except that using statistical concepts it is possible to quantify the risk that will be involved if the variability of the properties involved is known. In this sense, the method provides much more information than older methods. Figure 6 illustrates the concept as it was applied in a preliminary design study of the Dallas-Ft. Worth runways (Ref 37).

In that study the stresses expected under each of the design aircraft were calculated. A series of analyses were made to simulate various load transfer conditions for the pavement. In this case a series of computer runs were made to compare solutions obtained on the SLAB analysis programs developed at the University of Texas.

It is emphasized at this point that the stresses derived by any numerical solution are only simulations of the true stresses in the slab. Care must be taken in selecting the proper stress for design computations with a particular design method. Many methods give a maximum stress under the load, but this may not be the maximum in the slab or it may not be the one that characterizes the distress conditions observed in the field. Therefore, the designer should utilize the proper rationale in selecting stresses for use in design computations.

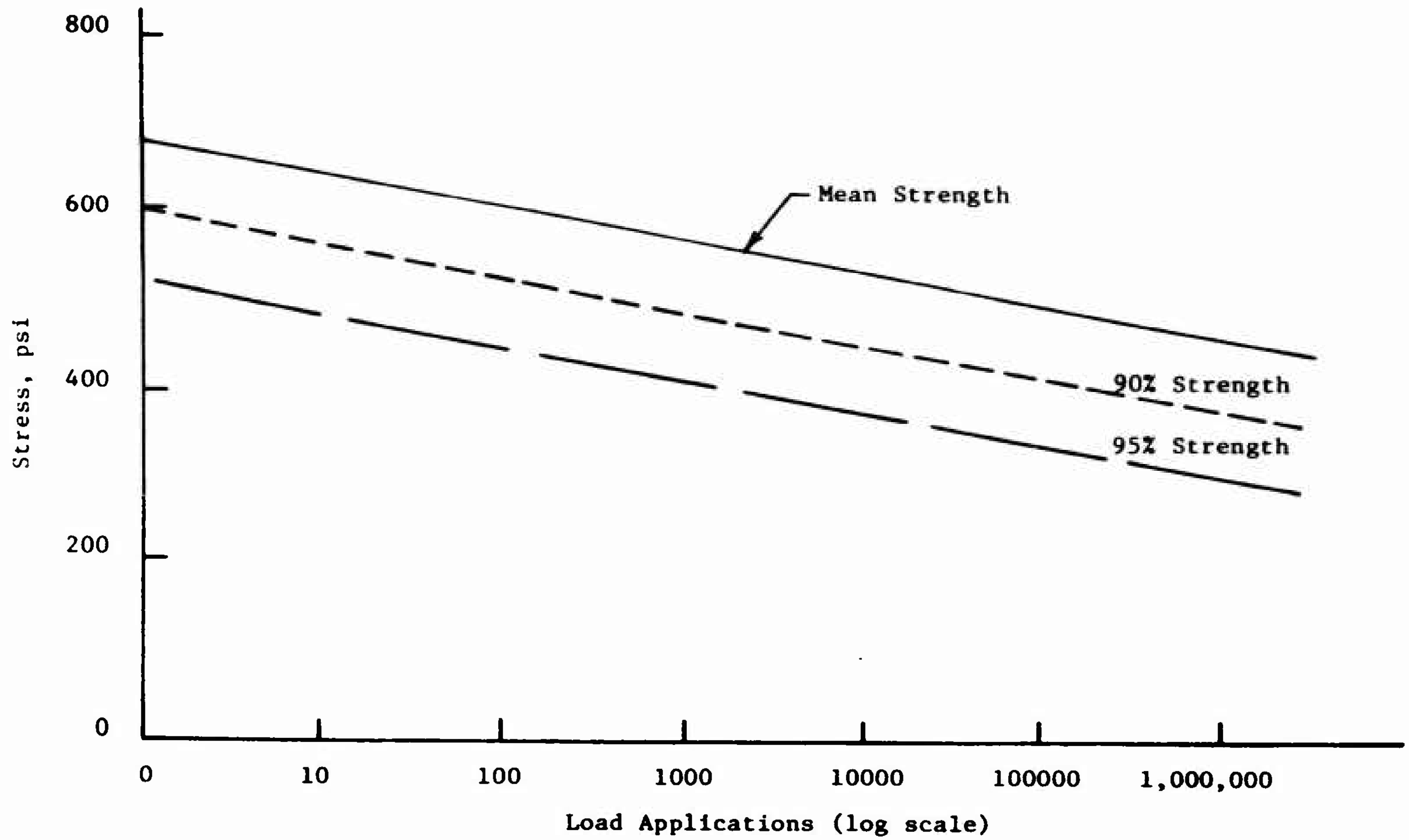


Fig 6 An example of concrete pavement fatigue curve used in confidence level analysis (Ref 37)

Figure 7 presents the pavement stress for the DC-8-63F aircraft and several loading conditions considered in terms of pavement thickness. This diagram was derived by plotting the stress calculated for each of the load conditions against pavement thickness. It may be noted that for a given thickness the stress varies depending on pavement type and stress type, i.e., top or bottom tension. In a safety factor type design the required pavement thickness would be obtained by entering the maximum allowable working stress (flexural stress/SF) on the vertical scale and projecting across to the proper curve and then down where the pavement thickness is determined. The heaviest aircraft is the controlling load and the other applications of any lighter loads would be assumed to have no effect on the pavement thickness determination under this method. It is obvious in this analysis that the selection of the safety factor is a key design decision since a slight change in its value will alter the results considerably. Although a value of 2.0 has been used in the past, the applicability of this value for unlimited load applications is questionable.

Fatigue Methods

While the safety factor method is based on arbitrary estimates, the fatigue method is based on the accumulated damage hypothesis of keeping the ratio of "anticipated load applications" to "permissible applications" for a given set of stress levels, to be equal to or less than one. The inputs required for such a solution are (Ref 37):

1. Load
    a. Magnitude
    b. Repetitions
2. Concrete Properties
    a. Modulus of elasticity
    b. Concrete flexural strength
    c. Variation in the properties
3. Subbase k-value
    a. Magnitude
    b. Variation

The allowable number of load applications must be determined by using a fatigue diagram such as shown in Fig 8. Continued work is needed to determine

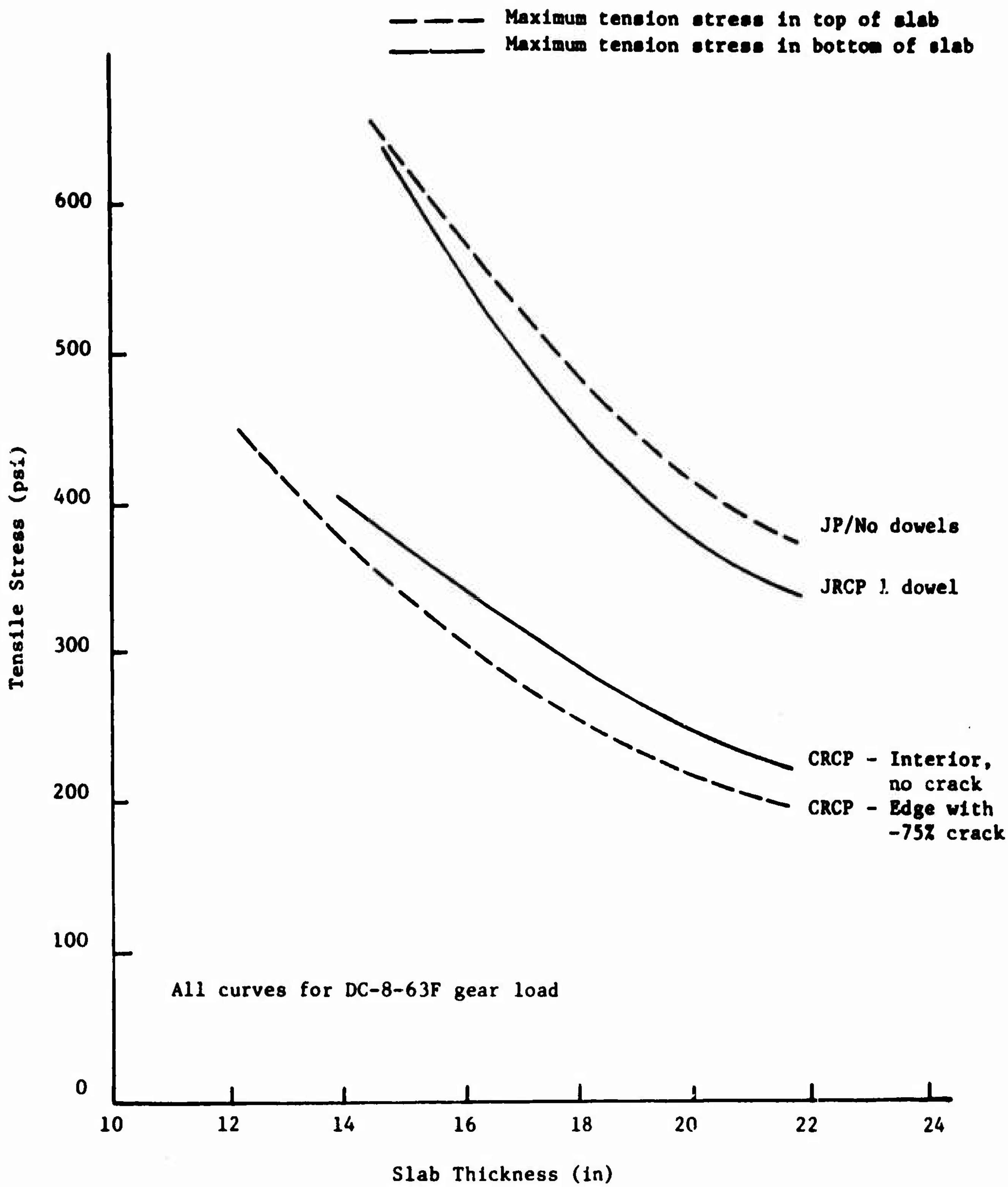


Fig 7 Possible design chart for safety factor method. (after Ref 37)

an exact fatigue-performance curve for airfield pavements. The material fatigue alone is not the controlling factor as illustrated for highways at the AASHO Road Test (Ref 28). The mean stress level of 700 psi is equal to that anticipated on the D-FW project and the slope of the line and standard deviations of the flexural strength in this example were derived from data taken on Runway 9R-27L at O'Hare Airport in Chicago, Illinois.

One basic assumption in such a method as stated previously is that the "life" is used up when the actual applications applied exceed the permitted number of applications for a given stress level. At that time, cracking is expected to occur, and the percent of the pavement area expected to experience cracking may be estimated, since the variation of material properties is considered.

For each of the design aircraft, the permissible number of applications is derived from Fig 8 for each confidence level and for the stresses for each pavement thickness. Using a fatigue computer program, the percent area expected to experience excessive distress for the anticipated loading is predicted for each pavement thickness. The expected distress is expressed as a percent of pavement area experiencing damage and is plotted as in Fig 9 for each pavement type and thickness.

The designer may enter the chart with a desired maximum level of damage at the end of the design life and determine the pavement thickness required. To use the figure, the designer enters with the acceptable damage level, projects vertically to the pavement type and then projects horizontally to the appropriate pavement thickness.

Figure 9 taken from Treybig et al (Ref 38) illustrates a slightly different way of looking at confidence level. In this case the data from the fatigue analysis are plotted in terms of confidence level. Using this type of curve, the designer can then enter at a fatigue damage summation of 1.0 and evaluate the slab thickness required for various confidence levels. Likewise, thickness can be used to enter the chart and a confidence level determined for each potential thickness. These data can then be converted to a thickness versus expected damage plot as shown in Fig 10 for several design lanes tried by Treybig (Ref 38).

Allowable Percent Damage

Perhaps the major weakness of this type of design method is that it is essential that some acceptable level of damage must be chosen by the designer

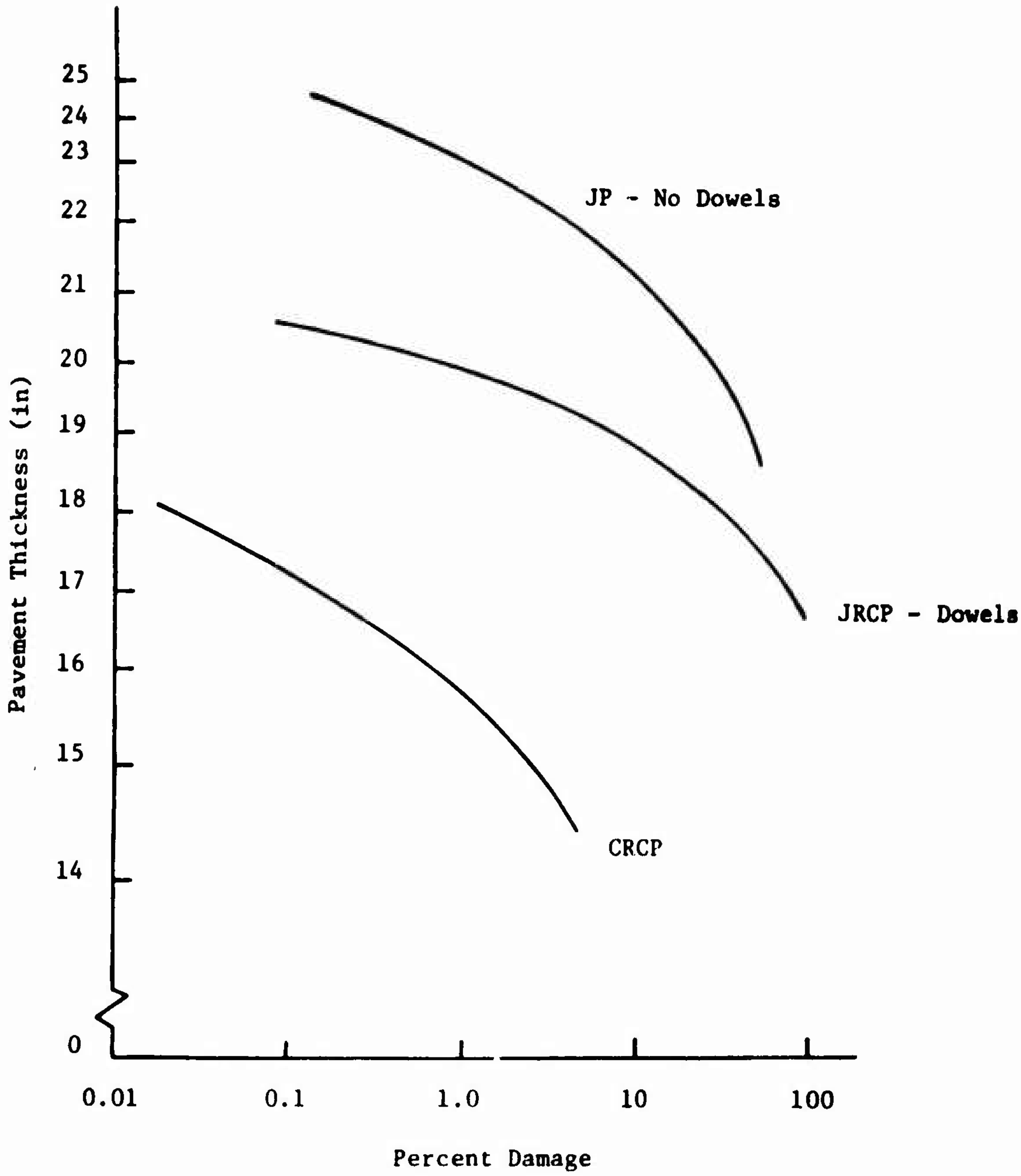


Fig 8 Damage-thickness curves for the fatigue method of design for two traffic cases. (after Ref 37).

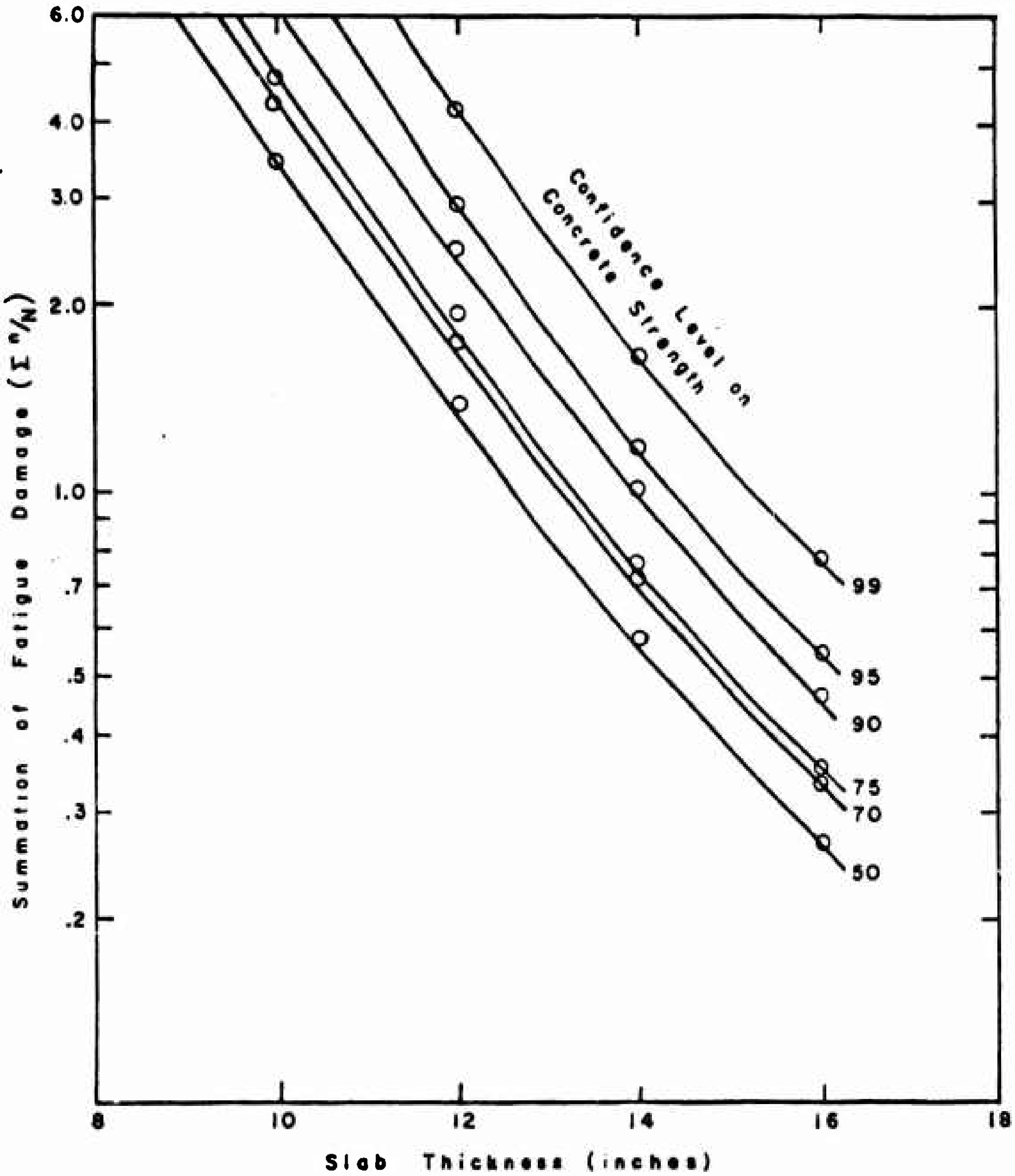


Fig 9 Graphical analysis of fatigue damage summation ratios and slab thickness (after Ref 38)

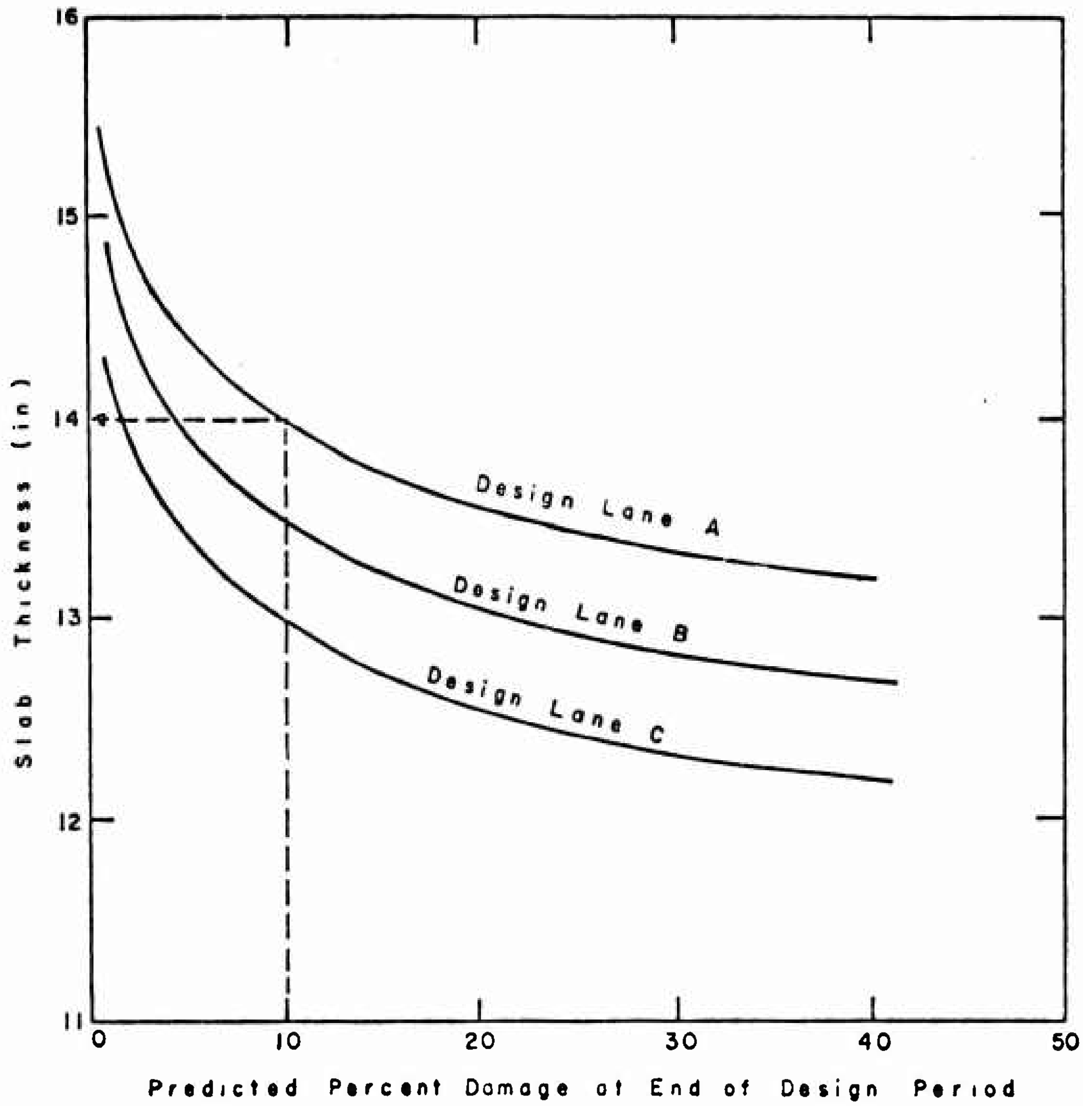


Fig 10 Illustrative thickness design curve
(after Ref 38)

and to date this must be done empirically. Ultimately, it will be essential that the percent damage be related to the "serviceability" of the pavement and the resulting user costs. The other major problem of this type of design approach is that a casual user of the method may mistakenly feel that it accounts for all possible variations, which it of course does not.

RELIABILITY CONCEPTS FOR PAVEMENT LIFE

A number of authors have contributed to the conceptualization of reliability for pavements. In this chapter we will outline some of the major work done to date in this field.

A General Reliability Model - Moavenzadeh

Reliability of pavements has been discussed in general terms for several years. Specific discussions of the subject have been given in references 6, 7, 8, and 9. One of the early groups to apply a meaningful mathematical evaluation to reliability include Lemer and Moavenzadeh at Massachusetts Institute of Technology (Ref 39). They present the concept of reliability as a design quantity in the context of systematic analysis of pavement systems. According to them, reliability is a measure of the probability that a pavement will provide satisfactory service to the user throughout its design service life. They point out that the prediction of reliability and its use in estimating economically efficient pavement life requires consideration of all aspects of pavement service life. These authors express the entire pavement design problem in terms of three principle measures of effectiveness: (1) serviceability, (2) reliability, and (3) maintainability. In this context they restate the definition of reliability,

> "Reliability is the measure of the probability that serviceability will be at an adequate level throughout the design service life of the pavement."

They point out that the future behavior of any engineering system is essentially uncertain. They define maintainability as a measure of the degree to which effort may be required during the service life of the pavement to keep serviceability at or above a satisfactory level. Since uncertainty is involved in all aspects of the pavement system including planning, design, construction, operation, and maintenance, Lemer and Moavenzadeh point out that the consideration of reliability is essential in the design of a pavement system. These uncertainties arise from lack of information and inability to predict the future. It is embodied in the assumption that must be made to

derive analytical models, the limited amount of data available from tests and the variable quality of the real world environment. These authors summarize this concept in Fig 11. They point out that factors which affect the degree of variation in pavement systems parameters have a significant effect on system reliability. They confirm that quality control in construction is a notable point of variability. They further point out that many people have failed to even recognize that these uncertainties are of serious consequence for future resource allocation predictions. By establishing reliability as a design parameter in pavement systems they attempt to take it into account in an explicit framework and to show how reliability considerations will interact with economics to influence decision making.

The authors do a good job of pointing out that since reliability is the probability of success or rather the probability that the pavement will resist the traffic and environmental loads applied to it throughout its design life, that the designer in order to evaluate the pavement's reliability must be aware of what the systems' possible modes of failure are and how they occur. In general for each failure mode, i, there will be an environmental load D_i placed on the pavement and a capacity of the pavement to resist that load R_i. The load D_i are determined by a set of environmental qualities (e_1, e_1, e_2, . . . , e_L). The pavement's response is determined by a set of systems characteristics (c_1, c_2, . . . , c_M). Then if there are N possible failure modes, failure is the condition in which one or more of the following inequalities is not satisfied;

$$\begin{aligned} D_1(e_1, e_2, \ldots, e_L) &\leq R_1(c_1, c_2, \ldots, c_M) \qquad (4) \\ D_2(e_1, \ldots, e_L) &\leq R_2(c_1, \ldots, c_M) \\ &\vdots \\ D_N(e_1, \ldots, e_L) &\leq R_N(c_1, \ldots, c_M) \end{aligned}$$

That is, if the demand on the system exceeds the ability of the system to resist that mode of failure, such failure would then occur.

Combined Failure Modes. Parenthetically, I would like to add here that not only must the pavement resist each mode of failure independently, for example, excessive cracking alone or excessive rutting alone may cause

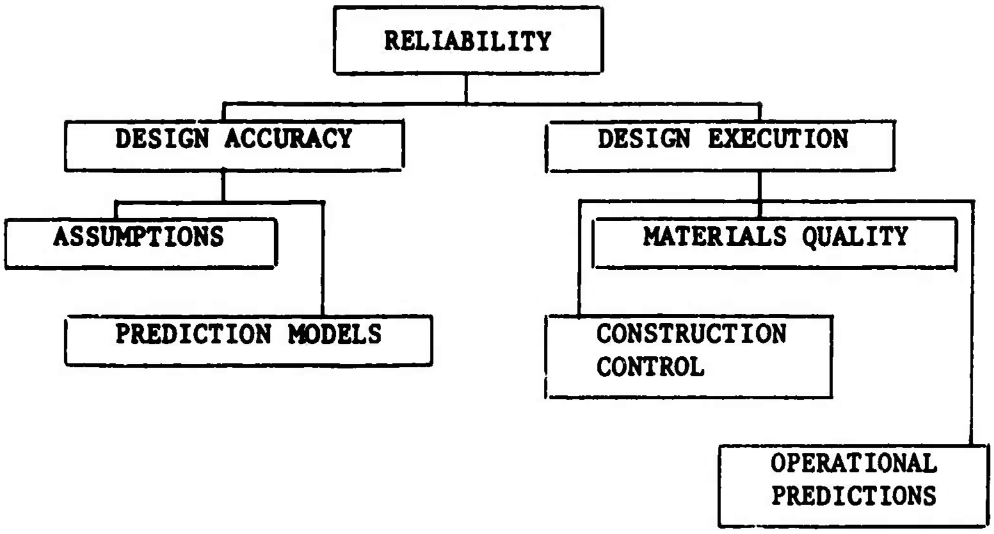


Figure 11 Components of reliability (after Ref 39)

failure; but most failures actually occur as a combination of distress modes, each of which alone has not reached a failure level. This is best illustrated by the present serviceability index which combines various levels of rutting, roughness, cracking, and patching with appropriate weighting functions.

For each failure mode, a model - theoretical or empirical or some combination thereof - is needed to determine how this failure would occur, i.e., how the pavement behaves under load. Theories of stress distribution in pavement systems are examples of such models for deformation, as are the equations produced by the AASHO Road Test for subjective evaluations of ride. That is, these models give a functional relationship between service loads and a parameter that is important to service quality, which is in these cases service deflection or riding quality. According to Lemer it is then possible to describe failure in terms of some maximum or minimum acceptable value of the parameter, which in turn defines the service load that is most likely to result in that value.

The application of these models (which may be uncertain) to data on the system environment characteristics that are probalistic (and which are uncertain) permit the calculation of the reliability R, which is the probability that all of the previous inequalities are true. Thus, R = P (no failure) = $P(D_i \leq R_i)$, where $i = 1, \ldots, N$.

Lemer and Moavenzadeh then show a major remaining task is to analyze the probabilities that individual failure modes will occur and to combine these failure modes to compute reliability. It is difficult to attain the initial estimates of failure probabilities because of the complexity of the physical processes involved. It is often impossible to arrive at closed-form mathematical statements of the probabilities involved.

In order to get around this difficulty of arriving at closed-form mathematical statements of the probability the authors proposed in their paper to use simulation methods, particularly the Monte Carlo simulation techniques. For such techniques they supply input data in probabilistic form, usually gathered by experimentation by sampling a random distribution. Many times it is possible with simulation techniques to estimate the probability distribution. In some complex situations, this technique may be the most feasible way of estimating the probabilistic output according to the authors.

The authors next define reliability as a time-dependent parameter where the effects of one step influence the occurrence of the next item in the time series. Such condescendent activities are simulated by the Markov Chain process by the authors. In their paper the authors go on to make some preliminary calculations of a pavement design reliability problem using simulation techniques.

Subsequent to the work by Lemer and Moavenzadeh, Moavenzadeh et al attempted to apply the Monte Carlo simulation technique to a complex pavement design method for the Federal Highway Administration. Their task was greatly complicated by the fact that a viscoelastic mechanical subsystem was also to be used. However, little success was actually obtained in using the Monte Carlo simulation. Rather a modified technique was used because the computer time associated with the Monte Carlo simulation proved to be excessive and it proved to be impractical to apply the techniques in a real design sense (Ref 40).

Application of Reliability Concepts in Rigid Pavement Design

Ramesh Kher et al applied concepts of reliability to the systems analysis of rigid pavement design for the Texas Highway Department (Ref 19). In RPS1, a working pavement management system probability is applied in computing the design values of the following variables:

(1) flexural strength of concrete,

(2) modulus of subgrade reaction, and

(3) Texas triaxial class of the subgrade.

It is assumed that these properties in a large population of samples, if plotted against percentages of occurrences, will fall along a continuous probability distribution defined by a normal curve.

For this type of data, the probability that x will assume a value between x and x + dx is given by dP as

$$dP = \frac{1}{\alpha\sqrt{2\pi}} e^{-\frac{(x - \bar{\mu})^2}{2\alpha^2}} dx \qquad (5)$$

where $\bar{\mu}$ and α are, respectively, the universe mean and standard deviation. The integral of the above equation over all values of x is equal to unity. The integral can be solved by using an inverse error function subroutine in the computer. Kher, however, uses a modify procedure which he developed for

simplicity. In any event the computer searches out the necessary design probabilities for established confidence levels.

In practical terms the designer using RPS must decide upon a design confidence level and input this into his problem with other values including a variance or a coefficient of variation for flexural strength and for subgrade modulus of reaction.

Assume for example that a design confidence level of 95% is selected. Then the outputs of RPS will be a series of near optimal pavement designs all of which can be expected to perform at a 95% reliability level with respect to concrete and subgrade strength. Another way of saying this is that a 95% probability exists that premature failure will not occur due to unexpectedly low strength in the slab or subgrade.

The reliability analysis is imbedded within the Kher method and the designer-user does not get involved with comparing cost of increased reliability. This may be unfortunate since the user may not have an adequate perception of variations and their effect, but at the time the program was developed it was the only feasible way to handle the problem. In the future the possibility of printing out the designs for several confidence levels should be considered so as to enable the administrator to compare the cost of increased reliability and make an appropriate choice for final implementation.

Problems of Defining Pavement Reliability

After Lemer and Moavenzadeh defined pavement reliability as follows: "Reliability is the probability that serviceability will be maintained at adequate levels throughout the design life of the facility," Darter in Reference 41 points out that reliability may be defined in many ways such as in terms of the probability that the maximum tensile strength is not exceeded by the applied stress in the surface layer or some other such definition. A more complete definition is given by Darter and Hudson as follows:

> "Reliability is the probability that the pavement system will perform its intended function over its design life (or time) and under the conditions (or environment) encountered during operation. The four basic elements involved in this concept of pavement system reliability are probability, performance, time, and environment.
>
> Probability: Reliability is the probability of success that a system has in performing its function. There are significant variations

and uncertainties in prediction associated with all the models in any pavement design system, and therefore, the chance of success will always be less than 100 percent.

Performance: The degree to which a pavement performs its intended function is its "reliability." P, performance, (in this broad context only) can be defined in several ways in the pavement system with regard to serviceability, skid resistance, user delay due to maintenance operations, and cost. As used in this study, however, performance refers to the serviceability history of a pavement.

Time: This element is essential in the definition of reliability because the reliability of a pavement must consider its intended life or design period.

Environment: The environmental conditions include the operating circumstances under which the pavement was used. The environment that a pavement "sees" will greatly affect its life span, its performance, and consequently its reliability. Thus if the pavement's environment changes significantly from that which it was originally designed, it may not perform with the same reliability as before."

Darter points out that the basic cause of this unreliability is the inherent complexity of large systems along with a background of urgency and budget restrictions which are nearly always associated with their design and construction. For pavements this is further complicated since we are continually working within limited technological knowledge of the many factors involved. There is simply not enough time or money to examine and analyze each factor to be considered including the almost limitless variability of materials, environment, and traffic. Darter also reiterates the point previously made by Lemer (Ref 39) that the concept of reliability is greatly complicated by the various modes of distress and failure which can interact in pavement loss of serviceability and performance.

Setting Reliability Requirements

The reliability required of a pavement system is essentially to be determined by its users, the traveling public. Darter and Hudson (Ref 41) point out that there are several serious consequences from a pavement developing early or premature distress. These include high user costs due to delay as well as damage to vehicles due to roughness.

According to Finn (Ref 42), in pavement design, costs of providing an increment of increase in reliability should be balanced against the cost which would result if the reliability is not increased. Since failure or loss of serviceability does not carry with it the same great problem of loss of life that the collapse of a bridge or building might, the optional

reliability for pavement type facilities may be lower than in a typical "structure." In an attempt to define this adequately Darter et al (Ref 41) sketched a general relation which can be assumed to exist between reliability, R, performance, P, and cost, C, which are displayed in Figure 12.

There are two basic costs involved, facility cost and motorist cost as shown in Figure 12(a). Motorist costs are high at low levels of reliability and would decrease with increasing reliability while facility costs are low at low levels of reliability and increase as reliability increases.

Darter points out that as R increases, the C (facility costs) may increase at an increasing rate as 100% R is approached. This increase in R could result, for example, from factors such as:

1. use of better quality materials,
2. less material variation,
3. greater maintenance input, and
4. increase in pavement layer thickness (generally).

Figure 12(b) conceptualizes the relationship between performance and reliability. Performance will increase as reliability increases where performance is the integral of the serviceability-time plot shown in Figure 12(c). Therefore, according to Darter using the R versus C (facility cost) and R versus P relationship, a relationship between C and P may be established as shown in Figure 12(d). Additional treatment of this subject including the use of this concept and the quantification of reliability requirements as outlined by Darter are presented later herein.

Reliability Concept Developed by Darter

There are many input variables in the pavement design problem. Basically factors associated with the loss of serviceability or "failure of the pavement" can be divided into (1) traffic effects and (2) environmental effects. The relative effects of these two factors will vary with geographic location and traffic intensity. The FPS program or Flexible Pavement Design System considers only the effects of traffic loading and two major environmental factors, (1) swelling foundation soils, and (2) average temperature effects across the state. On the basis that traffic associated deterioration is the primary factor associated with Texas pavements, Darter developed a reliability analysis based on traffic load associated distress alone.

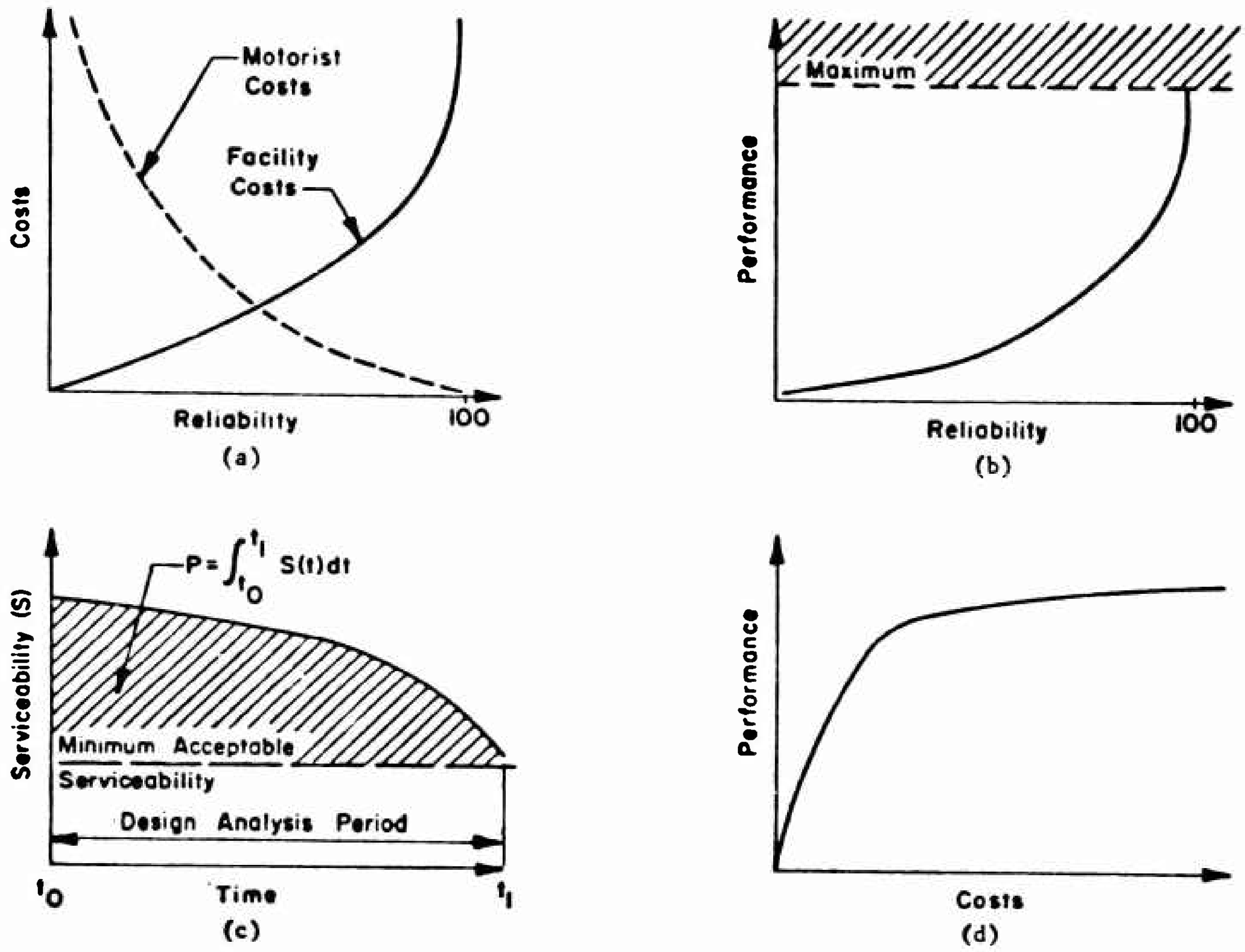


Fig 12 Illustration of relationships between reliability (R), performance (P), and costs (C). (after Darter Ref 41)

According to Darter et al the two basic parameters associated with predicting the life of a pavement section (considering only traffic-related failure) are the number of load applications the pavement can carry (or allowable applications) and the number of loadings that may be applied to the pavement. Both of these parameters are stochastic variables since the factors on which they depend are variable or stochastic. The reliability of a pavement section is determined from the basic concept that a "no-failure" probability exists when the number of load repetitions to minimum acceptable serviceability (N) is not exceeded by the number of load applications applied (n):

N = number of load applications that a section of pavement can withstand before minimum allowable serviceability is reached within a limited maintenance input,

n = number of load applications which are applied to a pavement section.

The number of load applications refers to 18,000-pound equivalent single-axle applications.

Reliability (R) is defined by Darter mathematically as the probability that N will exceed n, as shown by the following expression:

$$R = P\,[N > n] \tag{6}$$

where

P [ ] = probability that the event shown in the brackets will occur.

This statement is analogous to the statement that reliability is the probability that strength is greater than stress. Both N and n are stochastic variables and have a probability distribution associated with them, as illustrated in Fig 13.

The probability of n having a value of n_1 is equal to the area of the element of width dn , or to A_1 , as shown in Fig 13. The f(n) and f(N) are defined as the density functions of n and N , respectively:

$$P\left(n_1 - \frac{dn}{2} \leq n \leq n_1 + \frac{dn}{2}\right) = f(n_1)dn = A_1 \tag{7}$$

Since f(n) and f(N) are density functions, the probability that $N > n_1$ equals the shaded area under the f(N) density curve A_2 :

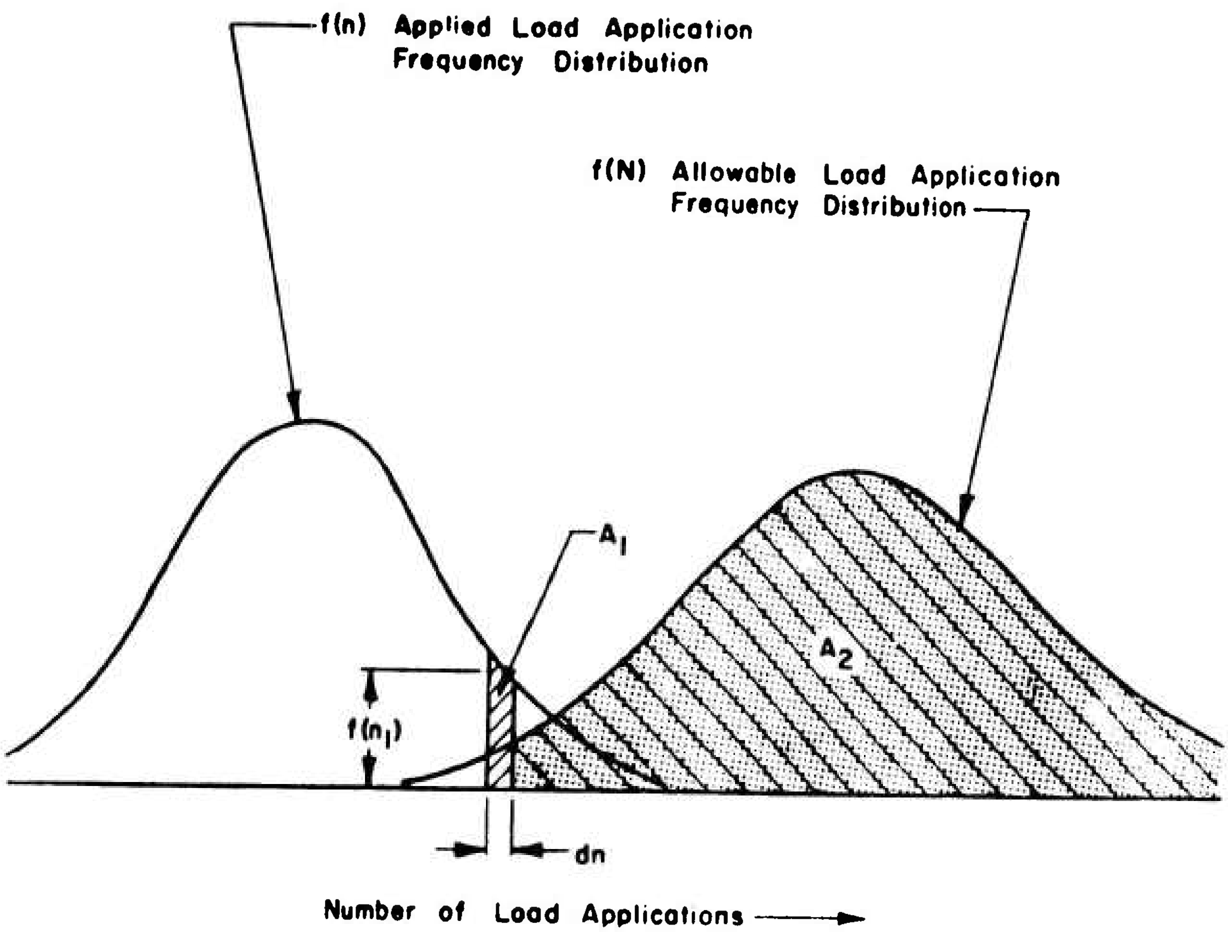


Fig 13 Illustration of allowable load application (N) distribution and applied load application (n) distribution. (after Darter Ref 41)

$$P(N > n_1) = \int_{n_1}^{\infty} f(N)dN = A_2 \tag{8}$$

The reliability R (i.e., the probability of no failure at n_1) is the product of these two probabilities.

$$P(n_1 - \frac{dn}{2} \leq n \leq n_1 + \frac{dn}{2}) \cdot P(N > n_1) \tag{9}$$

and

$$dR = f(n_1)dn \cdot \int_{n_1}^{\infty} f(N)dN \tag{10}$$

The reliability of the pavement structure is the probability that N will be greater than the possible values (over the range) of n . Thus, the basic equation

$$R = \int dR = \int_{-\infty}^{\infty} f(n) \left[\int_{n}^{\infty} f(N)dN\right] dn \tag{11}$$

where

$$\int_{-\infty}^{\infty} f(N)dN = 1, \tag{12}$$

$$\int_{-\infty}^{\infty} f(n)dn = 1 \tag{13}$$

Alternatively, an expression for reliability R may be obtained by considering that a no-failure probability exists when the number of applied load n remains less than the given value of N.

Equation (11) may be solved for exact answers if the distributions of N and n are normal or can be transformed to a normal distribution. The distributions of n and N are believed by Darter and Hudson to be approximately log normal, based upon the following check results:

(1) The N to terminal serviceability is similar to the number of applications to failure in a fatigue test. The fatigue life of asphalt concrete specimens under various loading conditions has been found to be approximately log normally distributed by Pell and Taylor and by Moore and Kennedy (Ref 43). The error in predicting the N to failure of the AASHO Road Test pavement sections was found to be approximately log normally distributed, (Ref 2).

(2) The n depends upon design ADT, percent trucks, axle factor, and axle load distributions. There are not adequate data available to verify that any of these factors is normally distributed. Since each of these depend upon several other factors, the error in prediction of each of these parameters would tend to approach normal according to the central limit theorem.

(3) Simulation of log N and log n using Monte Carlo techniques was used to give further data concerning the distribution of N and n. Values of the design parameters shown were selected from normal distributions using typical means and standard deviations and log N was calculated for 1000 trials. The same technique was used to obtain 1000 values for log n. The x^2 goodness of fit test, skewness, and kurtosis were used to test the hypothesis of normality. The assumption of normality could not be rejected at a level of significance of 0.05 for log N and log n. Using the same methods the design parameters were sampled for uniform distributions and the log N and log n were calculated. The hypothesis of normality was not rejected for log N, but was for log n at the 0.05 level of significance. Plots for log N simulated from normal and from uniform distributions are shown in Fig 14 and can be visually compared. There does not appear to be much difference between them.

Reliability is next evaluated by Darter considering log N and log n to be normally distributed (Note: All logarithms are to base ten.):

$$R = P[(\log N - \log n) > 0] = P[D > 0] \tag{14}$$

where

$$D = \log N - \log n$$

Therefore f(D) is the <u>difference density function</u> of log N and log n. Since log N and log n are both normally distributed, D will also be normally distributed. Function D is shown in Fig 15. Using bars above the expressions to represent their mean values, the following equation is written by Darter:

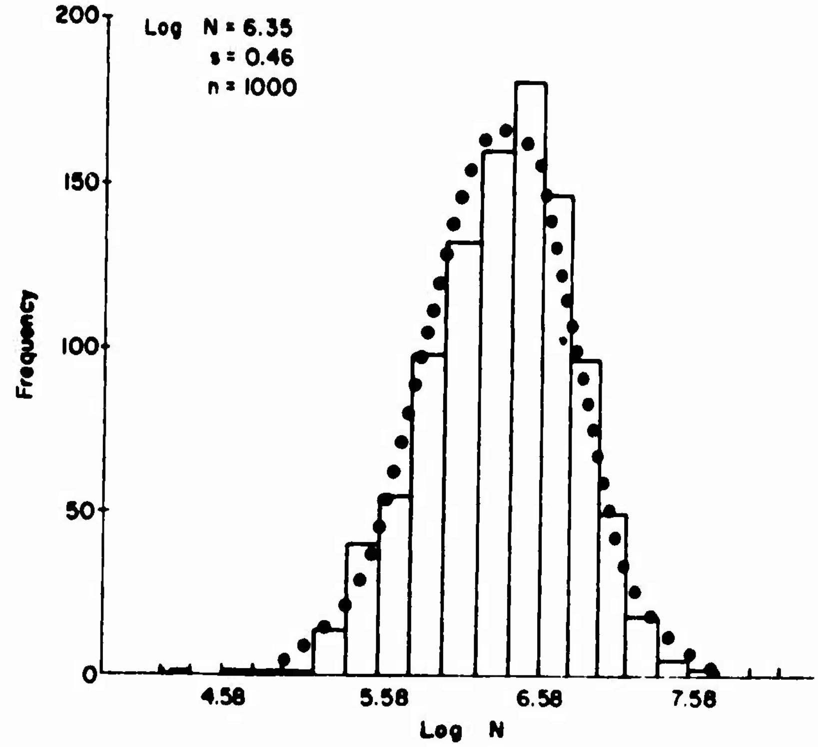


Fig 14 (a). Histogram of log N from simulation of design factors from normal distributions

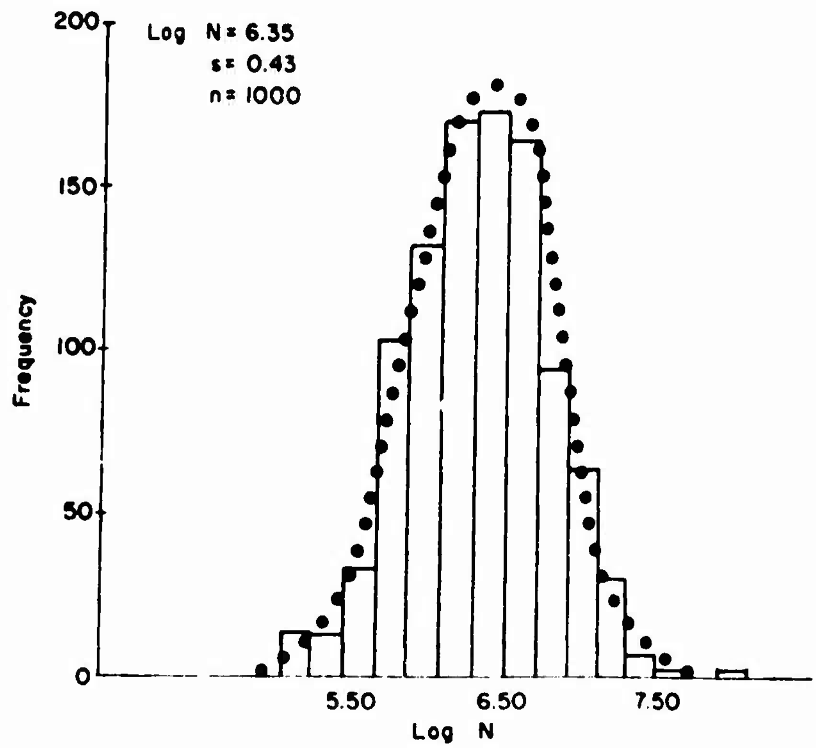


Fig 14 (b). Histogram of log N from simulation of design factors from uniform distributions (after Darter Ref 41)

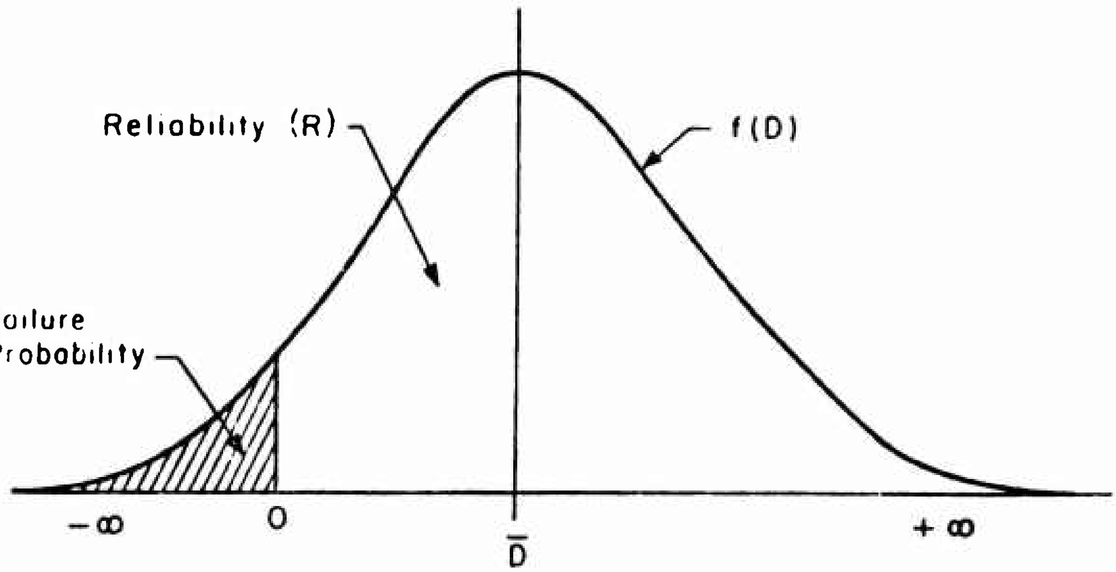


Fig 15 Difference density function (D = log N - log n).
(after Darter)

$$\bar{D} = \overline{\log N} - \overline{\log n} \tag{15}$$

The standard deviation of D is compute. as s_D by the following equation:

$$s_D = \sqrt{s_{\log N}^2 + s_{\log n}^2} \tag{16}$$

where

$s_{\log N}$ = standard deviation of log N ,

$s_{\log n}$ = standard deviation of log n

As shown in Fig 15, reliability is given by the area to the right of 0.

$$R = P[D > 0] = \int_0^\infty f(D)dD \tag{17}$$

or

$$R = P[0 < (\log N - \log n) < \infty] = P[0 < D < \infty] \tag{18}$$

The transformation which relates D and the standardized normal variable z is

$$z = \frac{D - \bar{D}}{s_D} \tag{19}$$

for

$$D = 0, \quad z = z_0 = -\frac{\bar{D}}{s_D} = -\frac{\overline{\log N} - \overline{\log n}}{\sqrt{s_{\log N}^2 + s_{\log n}^2}} \tag{20}$$

for

$$D = \infty, \quad z = z_\infty = \infty \tag{21}$$

The expression for reliability may be rewritten as

$$R = P[z_0 < z < z_\infty] \tag{22}$$

The reliability may now be determined very easily by means of the normal distribution table. The area under the normal distribution curve between the limits of $z = z_0$ and $z = \infty$ gives the reliability of a design as given from the following expression:

$$R = \frac{1}{\sqrt{2\pi}} \int_{-\frac{\bar{D}}{s_D}}^{\infty} e^{\left(-\frac{z^2}{2}\right)} dz \tag{23}$$

Darter gives an example for the calculation of reliability, assuming $(\overline{\log N}, s_{\log N}) = (7.100, 0.400)$, and $(\overline{\log n}, s_{\log n}) = (6.500, 0.200)$.

$$z_0 = \frac{7.100 - 6.500}{\sqrt{(0.4)^2 + (0.2)^2}} = -1.342 \tag{24}$$

From normal distribution tables the area from -1.342 to ∞ is 0.91. Therefore, the reliability R for this case is 91 percent.

<u>Reliability Considerations - Multiple Performance Periods</u>

In Texas the precise definition (Ref 41) of the reliability of a pavement design project for one performance period is the <u>expected</u> percentage of 0.20 mile sections along the project which maintain an adequate serviceability without the total maintenance cost exceeding a prescribed limit. When a pavement is designed for more than one performance period (e.g. stage construction), some complications arise as to handling the design situation.

A smooth overlay restores serviceability of the pavement to near the level it had just after construction. Observations in Texas show that various pavements exhibit widely differing performance characteristics during the second and succeeding performance periods after they have been overlaid. The time to failure of an overlaid pavement may be longer or shorter than the

initial pavement life. To further complicate the matter, the full decision criterion used for placing an overlay is not fully known. The reasons appear to vary widely from location to location throughout the state. The decision to place overlays is probably a function of such factors as (1) available funds, (2) traffic volume, (3) serviceability level, (4) areas of extreme localized failures, (5) distress manifestations such as cracking or spalling, and (6) even anticipation of the future distress.

The basic decision rule used by Darter for design is that the overlay will be placed when the expected 1 - R percent sections have reached minimum acceptable serviceability level. The overlay that is placed will be designed to last the next performance period with a probability of R.

Darter presented several cases formulated to illustrate the boundaries of the problem as observed in Texas (Ref 41).

> Case I. The pavement/subgrade system may be such that after a pavement section serviceability once falls to the minimum acceptable level, an overlay will only restore the serviceability for a brief time period. A pavement structure containing a cement-treated base that cracks badly is an example of this type. An overlay placed on this pavement will only maintain adequate serviceability for a short period of time. Assume that a pavement design strategy calls for three performance periods. The pavement is designed for R = 0.90 chance of success during each period and the overlay is placed when 1 - R sections reach minimum acceptable serviceability level. An analysis of the reliability involved is given in Table 1 for Case I. An expected 10 percent of the pavement sections reach minimum serviceability during the first period, 19 percent by the end of the second period, and 27.1 percent by the end of the third period. These probabilities are determined according to the assumptions that a section that has reached minimum serviceability cannot be restored to full capacity again, and that those sections that do not reach minimum serviceability have an R chance of success during the next period. The overall percentage of sections expected to succeed at the end of the analysis period is 72.9 percent.
>
> Case II. The pavement subgrade system is such that when a pavement section falls below the minimum allowable serviceability level and an overlay is placed, the section will have R chance of surviving the next performance period. Those sections that did not reach minimum serviceability during the period but were also overlaid will last throughout the entire remaining time of the design analysis period. A pavement that may have localized areas of failure caused by swelling subgrade or poor construction would be an example of this case. This pavement would normally be completely overlaid, and since many sections would be in good condition before the overlay, they would last throughout the rest of the design analysis period with the overlay. The reliability involved for this situation is shown in Table 1, where R = 0.90 and

TABLE 1. SUMMARY OF RELIABILITY CALCULATIONS FOR PAVEMENTS SHOWING CASE I, CASE II, AND THE SELECTED METHOD PERFORMANCE CHARACTERISTICS (after Darter Ref 41)

Situation	Performance Period	Expected Success (R)	Expected Failure
Case I	First	0.90	1.00 - 0.90 = 0.10
	Second	0.9 x 0.9 = 0.81	1.00 - 0.81 = 0.19
	Third	0.81 x 0.9 = 0.729	1.00 - 0.729 = 0.271
Case II	First	0.90	1.00 - 0.90 = 0.10
	Second	0.90 + 0.10 x 0.9 = 0.99	1.00 - 0.99 = 0.01
	Third	0.99 + 0.01 x 0.9 = 0.999	1.00 - 0.999 = 0.001
Selected Method	First	0.90	1.00 - 0.90 = 0.10
	Second	0.90	1.00 - 0.90 = 0.10
	Third	0.90	1.00 - 0.90 = 0.10

there are three performance periods. An expected 10 percent of the sections would reach minimum serviceability during the first period, 1 percent the next, and 0.1 percent the final period. The overall expected percentage of sections to succeed at the end of the design analysis period would be 99.9 percent.

The criteria given for Case I and Case II are believed to be the extreme ends of the spectrum of actual pavement performance. Therefore the following procedure was developed; it provides results that are between those described and is also an implementable procedure in the FPS program. This procedure is also felt to be closer to that actually occurring in the field than Case I or Case II.

Selected Method. The pavement/subgrade system is such that after an overlay has been placed, pavement sections that reach minimum acceptable serviceability and those that did not, previous to the placing of the overlay, both have R chance of surviving the next performance period. Therefore the pavement would show R percent of the sections succeeding at each performance period. This pavement may have combinations of Case I and Case II characteristics, but overall the average section that is overlaid at the time of 1 - R failures, will show R chance of lasting through the next performance period. An analysis of the corresponding reliabilities is shown in Table 1. The analysis is made as before, considering R - 0.90 and three performance periods. An expected 10 percent of all sections reach minimum serviceability during the first period, 10 percent by the end of the second period and 10 percent by the end of the third period. Therefore the overall expected percentage of sections to survive the design analysis period would be 90 percent. A pavement design strategy with none or several overlays would have the same reliability at the end of the design analysis period.

In summary, a pavement is designed by the selected method so that each performance period has an expected R percent of the sections maintaining adequate serviceability and the overlay is placed when the expected 1 - R sections have reached minimum acceptable serviceability. In reality, the number of sections to reach this level will vary from project to project as the expected percentage refers to all projects designed by the method. There is also some chance that the overlay itself will fail through improper construction and/or materials usage and therefore cause additional pavement distress not considered in this analysis."

Thus Darter and Hudson have treated the use of reliability in design as thoroughly as anyone to date. While these concepts have not been completely implemented they are rather complete.

Applying Reliability to AASHO Design Guides

In 1972 and 1973 Drs. Ramesh Kher and Mike Darter took some of the reliability concepts which had been developed for use by the Texas Highway Department and make a unique application of them to the AASHO Interim Guide for Rigid Pavement Design (Ref 44). First they studied thoroughly the known information on variance based on observed data.

The variance models were developed for the performance equation of the Guide to predict variation in pavement performance due to statistical variations in (1) traffic estimation, (2) flexural strength, (3) modulus of elasticity or concrete, (4) concrete thickness, (5) joint continuity, (6) foundation modulus, (7) initial serviceability index, and (8) lack of fit of the AASHO performance equation. Estimates of the variations associated with each of these variables were obtained by Kher by analyzing data from actual concrete pavement projects in Texas.

A new revised nomograph was developed to include, among other things, a scale for reliability and a scale for the "overall variance" determined by (1) the level of quality control exercised, (2) variations associated with design parameters, and (3) errors associated with traffic predictions. The variance models were also applied to determine the relative significance of the design factors associated with rigid pavement design and to quantify the effects of quality control on pavement performance.

Using the nomograph information they ran a series of problems to compare the significance of the results as presented in Table 2.

Revised Nomograph

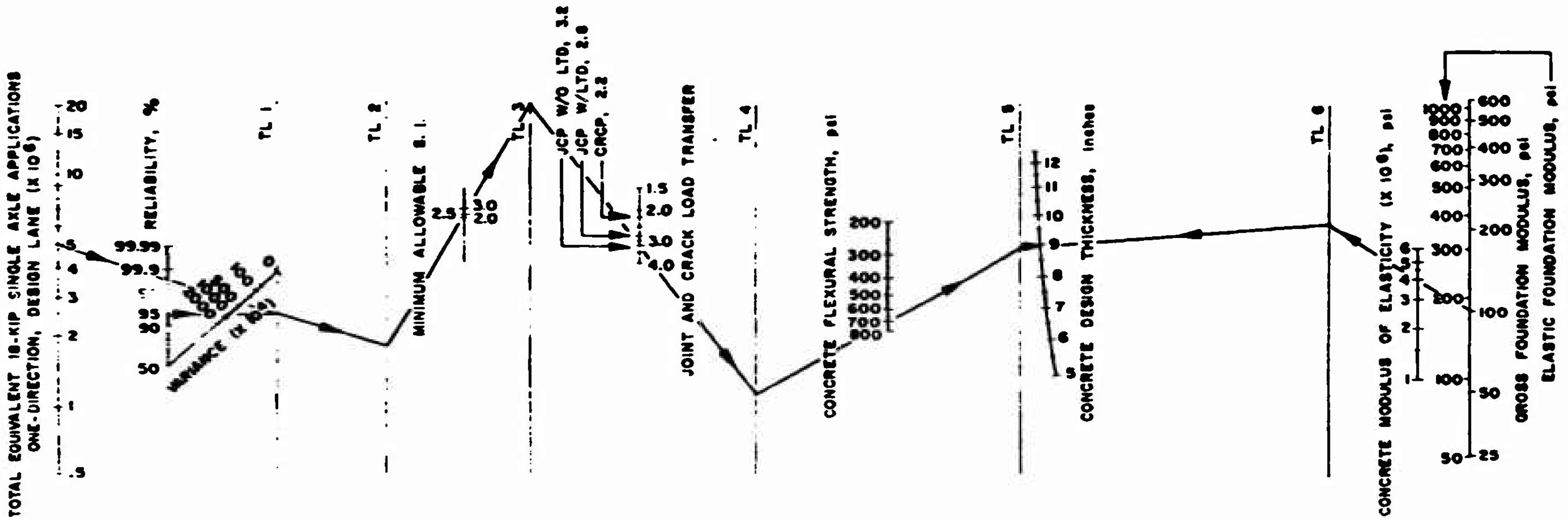
The AASHO Interim Guide nomograph as modified, is shown in Fig 16. The nomograph makes it possible to design a pavement thickness at any level reliability taking into account some of the uncertainties associated with various parameters. This is achieved by two scales shown in the nomograph, a variance scale and a reliability scale. The two scales are combined in such a way that the designed thickness is likely to last the required number of applications with the reliability for which the pavement is designed.

Variance (excluding the variance due to traffic) can be compiled theoretically using equations given by Kher and Darter. However, they developed Table 3 so that the summed variance can readily be obtained. This table was developed using variability characterization and judgment factors and represents average conditions of scatter in material properties and other parameters. They used the following values of variability to develop the table:

(1) Flexural strength, coefficient of variation = 10%,

(2) Concrete modulus, coefficient of variation = 10%,

(3) Concrete thickness, standard deviation = 0.3 inches,



EXAMPLE PROBLEM

Traffic = 5,000,000 single axle equivalent 18-kip applications
Variance = 1,000 (corresponds to average quality control)
Minimum Allowable Serviceability Index = 2.5
Joint and Crack Load Transfer Coefficient = 3.2 (JCP w/o load transfer device - LTD)
Concrete Flexural Strength = 700 psi
Concrete Modulus of Elasticity = 4,000,000 psi
Gross Foundation Modulus = 100 pci

REQUIRED CONCRETE THICKNESS

Reliability	90	95	99	99.9	99.99
Thickness, inches	8.6	8.9	9.7	10.4	11.5

Concrete thickness required by original interim design guide using working flexural stress of .75 × 700 = 8.75 inches (corresponds to 92.5 percent reliability)

Developed by
M. I. Darter
R. K. Kher
10 Aug 72

Fig 16 Nomograph for concrete pavement design at desired reliability level.

TABLE 2 SIGNIFICANCE ANALYSIS OF AASHO INTERIM DESIGN GUIDE, PERCENT EFFECT CONTRIBUTED BY EACH VARIABLE OF THE DESIGN EQUATION OVER A RANGE OF 32 REPRESENTATIVE PROBLEMS (after Kher and Darter Ref 44)

Problem Number	Flexural Strength	Concrete Modulus	Concrete Thickness	Foundation Modulus	Continuity Coefficients	Initial Serviceability Index	Lack Of Fit
1	24.1	.8	14.2	2.1	19.9	.3	38.6
2	26.0	.4	8.1	1.1	21.5	1.4	41.6
3	26.0	.3	8.5	.8	21.5	1.4	41.6
4	23.0	3.7	6.8	10.3	19.0	.3	36.9
5	23.8	2.3	9.5	6.4	19.7	.3	38.0
6	25.7	1.1	6.7	3.0	21.2	1.3	41.1
7	25.8	.7	7.4	2.0	21.4	1.3	41.3
8	27.0	1.3	14.3	3.5	10.5	.3	43.2
9	27.0	.9	15.8	2.4	10.5	.3	43.1
10	29.3	.5	9.2	1.3	11.4	1.5	46.9
11	29.3	.3	9.6	.9	11.5	1.5	46.9
12	25.6	4.1	7.6	11.4	10.0	.3	41.0
13	26.5	2.6	10.6	7.2	10.4	.3	42.4
14	28.9	1.2	7.6	3.3	11.3	1.5	46.2
15	29.1	.8	8.3	2.3	11.4	1.5	46.6
16	23.9	1.1	13.7	3.1	19.8	.1	38.3
17	23.9	.8	15.1	2.1	19.8	.1	38.3
18	26.2	.4	8.3	1.1	21.6	.5	41.9
19	26.2	.3	8.6	.8	21.6	.5	41.9
20	22.9	3.7	7.5	10.2	18.9	.1	36.7
21	23.6	2.3	10.3	6.4	19.5	.1	37.8
22	25.9	1.1	6.8	3.0	21.4	.5	41.4
23	26.0	.7	7.5	2.0	21.5	.5	41.7
24	26.7	1.2	15.3	3.5	10.4	.1	42.8
25	26.7	.8	16.8	2.3	10.4	.1	42.7
26	29.5	.5	9.3	1.3	11.5	.6	47.3
27	29.6	.3	9.7	.9	11.6	.6	47.3
28	25.5	4.1	8.3	11.3	9.9	.1	40.7
29	26.3	2.6	11.4	7.1	10.3	.1	42.1
30	29.1	1.2	7.7	3.4	11.4	.6	46.6
31	29.4	.8	8.4	2.3	11.5	.6	47.0
32	24.1	1.1	12.8	3.1	19.9	.3	38.6
Average	26.3	1.4	10.0	3.8	15.7	.6	42.1
Range	22.9 to 29.6	0.3 to 4.1	6.8 to 16.8	0.8 to 11.4	9.9 to 21.6	0.1 to 1.5	36.7 to 47.3

Concrete flexural strength, psi	Foundation Modulus, pci	JCP Without Load Transfer Devices: 6	8	10	12	JCP With Load Transfer Devices: 6	8	10	12	CRCP: 6	8	10	12
600	25	775	718	684	661	866	809	775	752	935	879	845	822
	100	779	722	688	664	870	813	779	755	940	883	849	825
	300	821	742	700	673	912	833	791	764	982	903	861	834
	600	918	777	719	685	1009	868	810	776	1078	938	880	846
700	25	775	718	684	661	866	809	775	752	936	879	845	821
	100	778	721	687	664	869	812	778	755	938	882	848	824
	300	810	737	698	671	901	828	788	762	971	898	858	832
	600	886	766	714	682	977	857	805	773	1047	927	874	843
800	25	775	718	684	661	866	808	775	752	936	879	845	821
	100	776	720	687	663	867	811	778	754	937	881	847	824
	300	803	734	696	670	894	825	787	761	964	895	856	830
	600	865	759	710	679	956	850	801	770	1026	920	870	840
900	25	776	718	684	661	867	809	775	752	937	879	845	821
	100	776	720	686	663	867	811	777	754	936	881	847	824
	300	797	731	694	669	888	823	785	760	958	892	855	829
	600	849	753	706	677	940	844	797	768	1010	914	867	838

TABLE 3 VARIANCES (10^4), FOR USE IN MODIFIED AASHO INTERIM DESIGN GUIDE NOMOGRAPH (IF VARIANCE DUE TO TRAFFIC IS CONSIDERED, IT MUST BE ADDED TO THESE VALUES) [illegible]

(4) Foundation modulus, coefficient of variation = 35%,

(5) Initial serviceability index, standard deviation = 0.3, and

(6) Continuity coefficient, standard deviation = 0.1 for JCP without load transfer units and 0.2 for CRCP and JCP with load transfer units.

The table does not contain initial serviceability index, and concrete modulus as variables since the sensitivity analysis showed the effects of variabilities in these parameters to be small.

Although Table 3 was developed from the best available data in connection with parameter variability and therefore can be effectively used for design, Kher and Darter encourage designers to develop similar tables using their equations to suit their own construction conditions and quality control.

The reliability scale presents a range of reliability from 50 to 99.9 percent. The designer can use any reliability in design. It was observed during the use of this and other similar systems by the authors that designers prefer to select this "R" number based on their own judgments and the importance of the highway facility under design. A thorough investigation of the old practice of using working stress as 0.75 times the flexural strength demonstrated that it corresponded to having reliability levels between 90 to 95% with the modified nomograph.

Variance of Pavement Performance Equations

Following the work of Darter, Holsen, et al (Refs 41, 45, 44) it is possible to separate the variance of pavement performance into its component parts using the partial derivative method. This assumes of course that you know the correct factors to use in the equation. All unexplained variation is absorbed in the lack-of-fit variance. Using the Texas model as an example the variance can be depicted as follows:

$$S^2_{\log N} = \left(\frac{\partial \log N}{\partial P1}\right)^2 S^2_{P1} + \left(\frac{\partial \log N}{\partial \alpha}\right)^2 S^2_{\alpha} + \left(\frac{\partial \log N}{\partial SCI}\right)^2 S^2_{SCI} + S^2_{lof} \quad (25)$$

where

$S^2_{\log N}$ = total variance associated with log N,

S^2_{P1} = variance of the initial serviceability index,

s^2 = variance of the temperature parameter,

s^2_{SCI} = variance of SCI of the pavement/subgrade system, and

s^2_{lof} = variance associated with the lack-of-fit of the performance equation.

Darter and Hudson presented approximate estimates of these variances, but the estimates were necessarily crude because of limited data. The input factor on which there had been the least amount of data gathered was the initial serviceability index.

Part of the variation associated with the performance equation is caused by the so-called lack-of-fit of the equation. The conventional way to quantify lack-of-fit error is to analyze "repeat" measurements, or measurements taken on similar "specimens." In evaluating the lack-of-fit error of the performance equation, this method constitutes a problem because of the inability of the pavement engineer to build exactly similar pavement sections. A different approach therefore was outlined by Holsen and he presents a method by which the lack-of-fit error of the performance equation can be evaluated.

This can be written as:

$$\text{Var}(\log M) = \text{Var}\left[\log\left(\sqrt{5 - \text{measured PSI}} - \sqrt{5 - P1}\right)\right] + \text{Var}[\log \alpha] + \text{Var}[2 \log SCI] + \text{Var}[LOF] \qquad (26)$$

Equation 26 is thoroughly explained in Chapter 3 of Holsen Report (Ref 45). Some of the inputs to Eq 26 are measured in the field, thus reducing some of the estimation (or pure) error introduced when all the parameters have to be estimated. The error introduced by lack-of-fit is the same in both cases, and by solving Eq 26 for the lack-of-fit term, the lack-of-fit of the performance equation at the design stage should be obtained.

The estimated load applications obtained from Holsen (Ref 45) should then be compared with the actual applied 18-kip single-axle load (m), which can be obtained from the Planning Survey Division, Texas Highway Department. Most likely there will be a difference between the two figures which herein is termed an "error." Both log M and log m are assumed to be normally distributed, and the error will therefore be normally distributed and have

a variance associated with it. The variance of the error is due to the variance of log M plus the variance of log m, and it can be written as:

$$\text{Var } [\log M - \log m] = \text{Var } [\log M] + \text{Var } [\log m] \tag{27}$$

or

$$\text{Var } (\log M] = \text{Var } [\log M - \log m] - \text{Var } [\log m] \tag{28}$$

There are now two equations for the variance of log M which are set to be equal, and the equation can be solved for the variance due to the lack-of-fit:

$$\begin{aligned} \text{Var } [LOF[= {} & \text{Var } [\log M - \log m] - \text{Var } [\log m] - \text{Var } [\log \alpha] \\ & - 2 \text{ Var } (\log SCI] - \text{Var } \log \left(\sqrt{5 - \text{measured PSI}} \right. \\ & \left. - \sqrt{5 - P1} \right) \end{aligned} \tag{29}$$

Data for Quantifying Lack-of-Fit

A considerable amount of data must be collected in order to quantify the lack-of-fit error associated with the performance equation in FPS.

A study of in-service flexible pavements that are in their first performance period is needed. Holsen discusses the data needed, factors to consider in the study, and how to obtain the necessary data. For Texas the factors include:

(1) present serviceability index for every consecutive 1200-foot section (possibly each 0.2-mile) along two or three miles of every project,

(2) surface curvature index, measured with Dynaflect, taken for every 100 feet to get good estimates of mean SCI within each 1200-foot section,

(3) estimate initial serviceability index for each section for which such information is not available,

(4) total number of 18-kip equivalent single-axle load applications since construction (estimated by Planning Survey Division, Texas Highway Department), and

(5) temperature parameter for the district in which the project is located.

Since the objective of such a study is to quantify the lack-of-fit of the performance equation used in the Texas Flexible Pavement Design System, the study should include a spectrum of all types of flexible pavements in the State of Texas.

Data Feedback Systems

In reality the problems outlined by Holsen beset the entire pavement field. There is a dirth of realistic data from which to actually check the models we are using. Inevitably the profession seems to feel that "we do not have time to collect data on existing systems." Yet it is only by comparing the predicted performance of various pavements with the projected performance life from various models. These data must include all of the pertinent factors which affect the pavement performance.

Ultimately an important step which must be taken in developing rational pavement design is the development of adequate feedbacks data systems involving important pavement variables.

Application of Reliability to SAMP 6

SAMP 6 is a "Systems Analysis Method for Pavements." The initial versions of the program were developed by Hudson and McCullough for NCHRP (Ref 20). It was subsequently modified and put into use by Lytton and McFarland (Ref 50). This program uses basic reliability inputs similar to FPS (Ref 41) and the concept is described by the authors as follows:

> Two reliability variables are input: a coefficient of variation and a confidence level indicator. The designer is required to furnish various stiffness coefficients, soil support values, and a regional factor all of which have to be estimated on the basis of field experience and some lab tests. None of these factors can be determined directly from a lab test; however. Despite that fact, each designer with some experience using the design method contained within the AASHO Interim design Guide (3) has some idea of within what accuracy he knows each of these variables. The coefficient of variation tells within what percent of the average he is sure that about 70 percent of all values he observes will fall. The confidence level indicator allows the designer to choose how certain he wants to be that the pavement he is designing will last for at least a minimum period of time before the first overlay and between overlays. The confidence levels that can be selected within the program vary between 50 percent and 99.9 percent.

In the appendix of that report Lytton and McFarland treat the pavement reliability condition in detail. Although no direct reference is cited, the

work is remarkable similar to that by Darter (Ref 41) previously discussed. In Appendix A to their report, Lytton and McFarland present a method of estimating the several components of variance similar to those sorted out by Darter. The process in both cases assumes that there is no correlation or co-variance relationship of terms to be considered in the analysis. This is probably incorrect but it does make it possible to estimate the needed variance.

In practice the variance equation obtained is used by Lytton as follows:

> "This equation appears in sub-routine PVPY which amplifies n, the predicted number of 18-kip equivalent loads that would occur in time t by a factor which depends upon the desired reliability. This amplified number is then compared with the "failure" condition, N, to determine whether the reliability condition has been met.

While the mathematics of these developments seems valid there are likely additional factors and co-variance terms which are unknown and have therefore been omitted. As pointed out above, however, these terms are being used in SAMP 6 on at least a trial basis in Kansas, Florida, and Louisiana.

SELECTING A LEVEL OF RELIABILITY FOR USE IN DESIGN

Traditionally we have come to use reliability in a very precise sense and at numerical levels which imply a great deal of confidence. For example we may read a statement in the newspaper or a report as follows:

Statement 1

> ". . . The reliability of the space capsule has been increased from 99.9% to 99.999%."

This implies on the one hand that the author knew precisely that the space capsule had exactly 1 chance in 1000 of failing under the conditions which existed for the statement. Secondly some change was made to the capsule which changed the odds of failure such that there is only 1 chance in 100,000 of failure for the subsequent conditions.

There is serious doubt that such exact standards can conscientiously be applied for such statements. For this to be true not less than 100,000 tests (and preferably at least twice that many) would need to be conducted of the full scale space capsule under the stated conditions to show that only 2 out of 200,000 actually failed.

Obviously complex systems such as a space vehicle, or a pavement system cannot economically be tested so widely. Rather components of the system are tested and calculations and computer simulations of the resulting system are used to generate estimates of reliability.

Qualitative Statement of Reliability

It should be understood then that the author in the example above probably meant to say something about as follows about his original space vehicle.

Statement 2

> "After exhaustive testing within our available budget we estimated that space capsule version 1 was very reliable. So reliable in fact that its chances of success we estimate to be better than 99/100 (or one failure per 100), therefore we say the next logical level of 999/1000 or 99.9% reliability. We can't prove this level to be exact of course because we can't afford to test 1000 capsules, but we are 'very very' confident of our product."

Now with this type of background in mind a significant improvement might have been made in the capsule (perhaps even after the second or third

vehicle failed in flight, e.g. 1 in 2 or 3). The author wishes to convey to the reader that this significart change has been made. His complete statement might have to read as follows:

Statement 3

"I want you to know that we have achieved a breakthrough in space vehicle design. We have solved significant problems found in version one. Thus version two is much much better than the older model. Since published data have already implied that it was 99.9% reliable, I have no choice but to use a much higher reliability (in this case I'll use 99.999%) to show you how much better I 'think' it is."

He first wrote the following statement but the public relations group rejected it:

Statement 4

"We originally estimated our space capsule to have a reliability of 99.9% (we really meant less than one expected failure per 100). However since capsule number three failed in flight we probably should revise that reliability to 66% or 80% (say one failure in 3 or 5). However we now have a revised space capsule and we feel it correctly has a reliability of 99.9%."

Of course both statements 3 and 4 are more realistic than statement 1 but they imply "doubt," which sometimes causes the reader to reject a concept altogether, therefore he uses Statement 1 as shown above for emphasis.

In reality most statements of reliability should be qualitative--not quantitative. This does not mean that they are not useful--they are. It merely means that we often do not have an adequate basis for making predictions as precise as 1 in 100,000.

Due to the logrithmic failure often observed for complex devices or systems, a decade change in reliability (e.g., 1/10 to 1/100) is often considered the minimum useful change. A typical scale might be 50%, 75%, 90%, 99% 99.9% . . . etc. The reciprocals give us the chance of failure as 1/2, 1/4, 1/10, 1/100, 1/1000.

Use of Reliability

Keep in mind then when establishing reliability that the scale can be useful but also misleading. We should continue to use a quantitative scale but we should understand its limits and when new data seem to indicate a reduction is needed for a previous estimate, professional pride should not keep us from downgrading the original estimate.

Finally as experience builds up and we do obtain real data on which to estimate revised reliability with greater confidence, we should do so and indicate the reason for the change.

Calibrating a Reliability Scale

Most of this report has been given over to defining reliability and developing a method for using it in pavement design. Let us assume then that we have an effective reliability model such as that used by Darter (Ref 41) or Lytton (Ref 50); or that we are using a confidence level method such as outlined by Treybig (Refs 35, 36). What reliability or confidence level should we adopt for design? The answer depends on three major factors.

(1) The class of service or quantity of traffic to be carried on the facility,

(2) The cost or loss associated with an unexpected failure of the pavement.

(3) The cost or other factors associated with providing an increased increment of reliability.

In other words the problem is one of trade offs. The cost of increased reliability vs the resulting saving to the user. Darter hypothesizes this relationship in Fig 12 herein.

In view of these facts, methods have from time to time been espoused for quantifying design judgment. Basically the concepts are all similar. It is desired to capture the decision making experience of several key administrators and quantify this process for future use. Implicitly this is a difficult task because many successful leaders (1) do not know what makes their decisions valid or (2) jealously guard the uniqueness that they have, which has permitted their rise to the top of the heap.

Design Judgment

All design decisions ultimately lie in the hands of the owner, the boss, or the "chief," that is the person with authority to commit funds and resources to a project. Keeping this in mind we realize that many decision criteria for pavements remain qualitative and judgmental as outlined by Fig 17 taken from the NCHRP Report by Hudson and McCullough (Ref 20).

I therefore know of no successful application of such techniques as Bayesean analysis in this field, but the rewards of success seem to justify continued research in the area. More quantitative work is needed in this area of research.

In the intermediate term however, the question arises in the pavement designer's mind "What does a 90% reliability or confidence level mean to me?" Then a corollary question, "How much better would a reliability of 99% be?" The Texas pavement research team attacked this problem in Project 123 (Refs 19, 41, 44). In summary the approach involved taking several typical design examples for standard materials often used in the state. For these conditions the designers felt strong confidence in the existing structural design method or subsystem. For these relatively standard conditions a series of system designs (FPS) were run at various reliability levels. A conference was then held in which a number of the designers selected a thickness design from the list available without knowing the associated reliability level. Amazingly there was a general consensus on reliability for like conditions even though a code was used for "reliability level" and not a number (such as 99%) to confuse the issue.

Recommended levels of design reliability for the FPS-11 program as contained in the user's manual are shown in Table 4. From six possible levels, only three levels were recommended by the "designers"; 95, 99, and 99.9. The decision criteria consist of (1) whether or not the project is located in an urban or rural area and (2) whether or not the highway will be operating at less than or greater than 50 percent capacity throughout the analysis period. The higher reliability is associated with the urban area location and with the traffic volume greater than 50 percent of the capacity.

The results obtained from the 12 projects were analyzed further and additional recommendations were developed to supplement those contained in Table 4. Proposed criteria were listed by Darter as follows: (Ref 41)

(1) number of 18-kip equivalent single-axle loads;

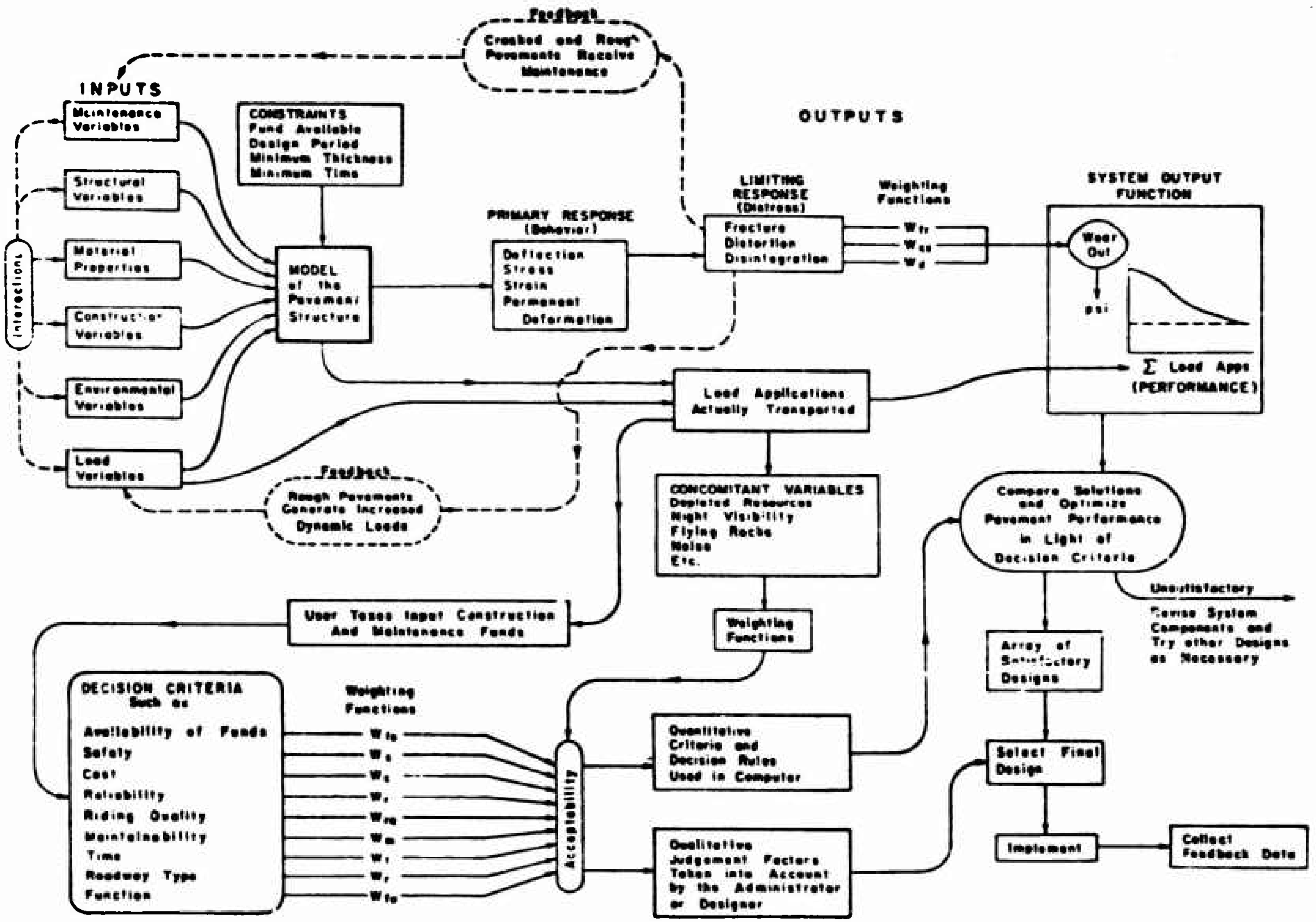


Fig 17 Block diagram of conceptual pavement system.
(Ref 20).

Table 4. Guidelines for Selecting the Design Confidence Level (from FPS User's Manual)

	The highway will remain rural throughout the analysis period	The highway is or will become urban before the end of the analysis period
The highway will be operating at less than 50% of capacity throughout the analysis period	95 or 99	99.9
The highway will be operating at greater than 50% of capacity sometime within the analysis period	95	99 or 99.9

(2) the degree to which traffic congestion will be a problem during overlay operation, which depends upon traffic volume and available detours;

(3) highway functional classification, arterial or collector; and

(4) location of highway, urban or rural.

The resulting set of criteria are shown in Table 5. The design reliability level can be selected using Table 5 if the criteria are known. If the road is in an urban area, according to Darter, the higher reliability level should be used wherever alternate levels are given. These recommended design reliability values are tentative of course and only experience with the FPS design or similar system will provide verification and improvement.

<u>Selecting Optimum Design Strategies Considering Several Reliability Levels</u>

In practice pavement design is usually carried out at a single level of reliability by applying a specified safety factor to one or more design inputs. The consideration of designs at various levels of reliability has been done in various ways in the past, however, but not on a formalized basis. The consideration of design strategies at several levels of reliability is important because of the large differences in such factors as user costs and pavement performance at different reliability levels.

As pavement design reliability increases, pavement performance on the average increases, as is illustrated in Fig 18 for two design strategies (from Darter, Hudson and Haas Ref 49). The mean expected performance curve is higher for the strategy with greater reliability. The general conceptual relationship between pavement reliability and performance for a given project situation is shown in Fig 19. At every reliability level there exists a range of alternative designs, as has been discussed, and each of these designs exhibits a certain performance. The range of performance of these alternatives is illustrated in Fig 19.

As the level of reliability increases, facility costs increase, and user costs decrease. The increase in facility cost with increased reliability is due to such factors as use of better quality materials, greater maintenance, and an increase in pavement layer thickness. In other words,

Table 5. Recommended Design Reliability Levels for FPS Program (after Darter Ref 41)

Traffic Handling situation	Functional classification	Equivalent 18-kip single-axle loads: <500,000	500,000 to 2,000,000	>2,000,000
Satisfactory	Collector	95	95 or 99	99
Satisfactory	Arterial	95	95 or 99	99 or 99.9
Some Problem	Collector	95	95 or 99	99
Some Problem	Arterial	95 or 99	99	99 or 99.9
Considerable Problems	Collector	95 or 99	99 or 99.9	99.9
Considerable Problems	Arterial	99	99 or 99.9	99.9

*Note: If pavement is located in urban area, use higher reliability level wherever range is given.

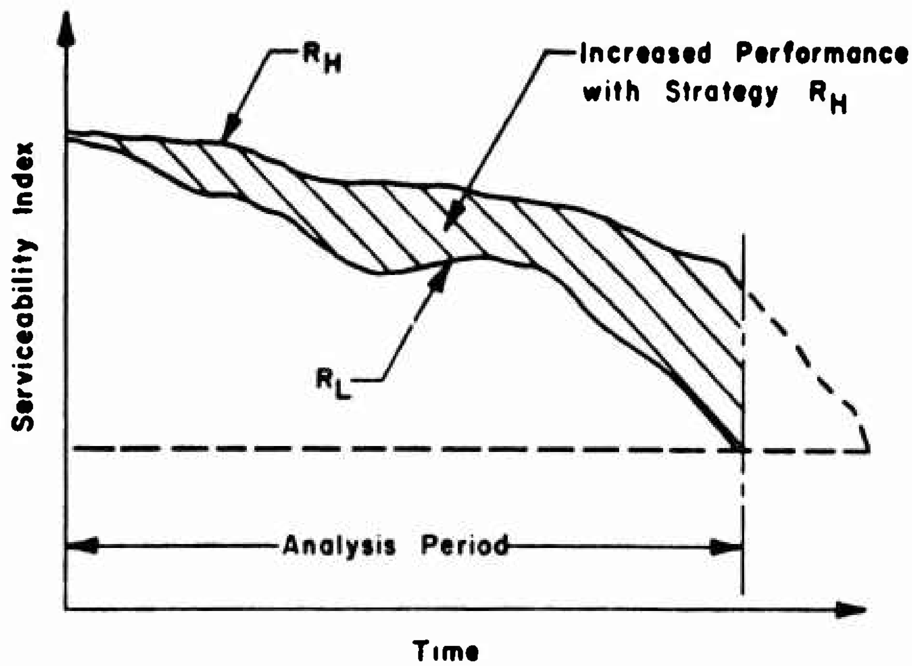


Fig 18 Illustration of mean expected performance curves for design strategy with relatively high reliability (R_H) and low reliability (R_L). (Ref 49)

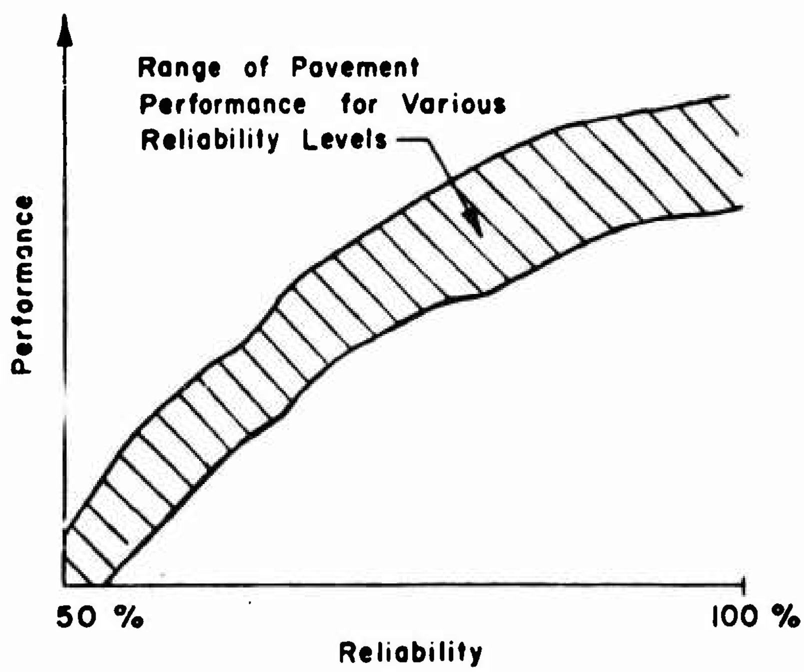


Fig 19 Conceptual relationship of reliability and performance. (Ref 49)

to provide a pavement that will have a greater chance of performing as desired, the required facility costs will be higher and the resulting performance will on the average be superior. A general relationship between pavement reliability and facility costs may be as illustrated in Fig 20. There will exist a range of alternative designs at each reliability level as illustrated by the band width in Fig 20. On the other hand, as reliability increases, the pavement user costs should, in general, decrease since the performance is generally higher resulting in decreased user delay, accident, and vehicle operation costs.

The various interrelationships can generally be summarized by the plot in Fig 21 developed by Darter et al by summing ordinates of user and facility costs for each level of reliability. According to the authors:

> "The shape can be logically explained by using three hypothetical design strategies, i , j , and k , for a given project, as is indicated on the plot. Strategy i represents a design at a low level of reliability or low safety factor (not less than 50 percent, however). Such a strategy would have a small chance of performing as expected and would on the average exhibit rather low SI throughout its design life and have several unexpected pavement failures associated with it, requiring rehabilitation repairs. The facility costs may be relatively low, but user costs would be relatively high because of a low serviceability level of the pavement over its design life. This results in high user delay and vehicle operation costs due to excessively rough pavements.
>
> Strategy k represents a pavement design at a relatively high level of reliability where the corresponding facility costs are very high and user costs are low because of a high performance level of the pavement. This strategy represents very heavy initial construction and minimal or zero maintenance.
>
> Strategy j represents a pavement somewhere between the extremes of i and k . This design strategy represents a design that has facility and user costs which combine to give an overall minimum total cost. The level of performance and reliability expected is between that of i and k .
>
> The reliability associated with design strategy j represents the level that would generally give a minimum total cost for the project. However, other factors that must be evaluated are performance and reliability. Does the expected performance curve give an adequate level of service to the user? This must be judged by the engineer from previous experience and from the magnitude of associated user costs."

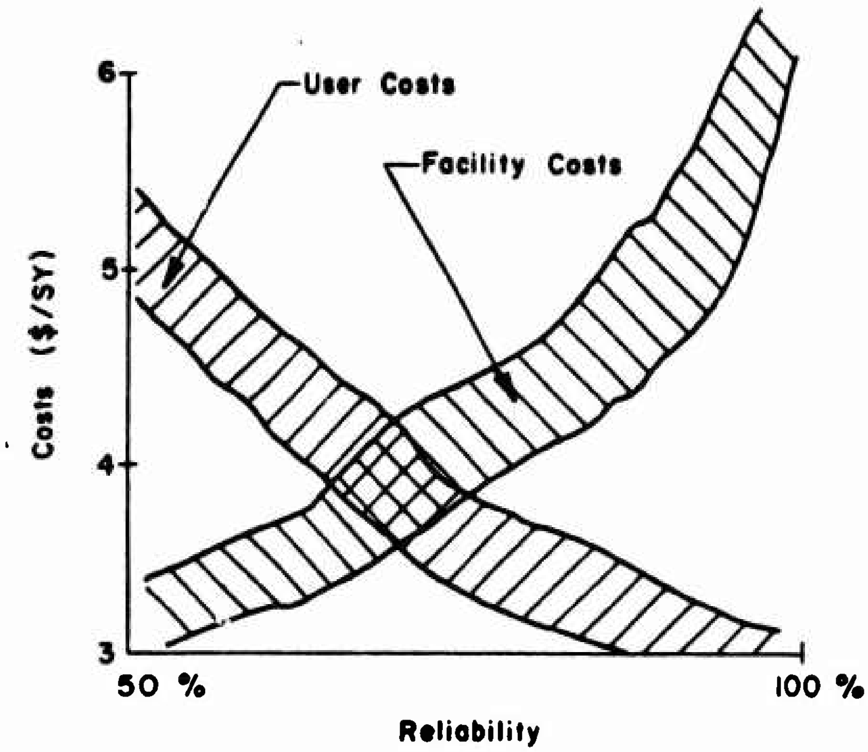


Fig 20 Relationship between pavement costs and reliability. (Ref 49)

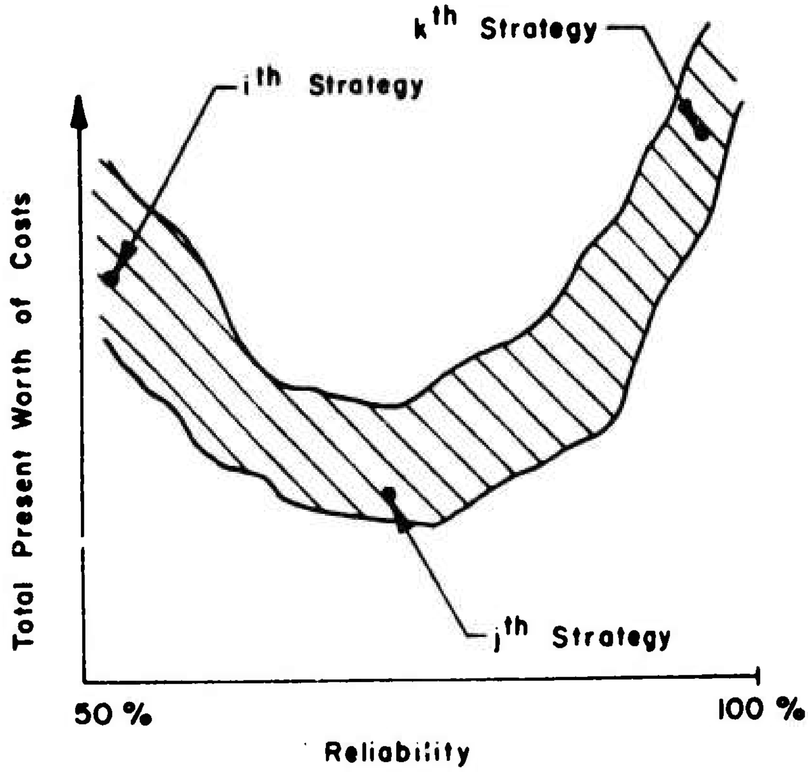


Fig 21 Relationship between total pavement costs (user plus facility) and pavement reliability. (Ref 49)

SUMMARY

There is no point in an extensive summary of a report such as this which is itself a summary of the state-of-the-art. Rather we will recapitulate the items covered herein.

The pavement system is pointed out as a methodology for defining and considering variability and reliability in pavement design. Many people are becoming aware of the effects of variability. In general this has been considered in design by

(1) improving quality control in construction as much as possible.

(2) provide for increased design confidence by establishing design confidence charts.

(3) define reliability and solve the system equations for increasing reliability.

In all cases increased quality, confidence, or reliability also results in a corollary increase in cost. It is necessary to quantify on equal or increased benefits to justify the higher reliability. I believe this can be done but little work is evident to date.

It is recommended that research continue to provide better information relative to design reliability and levels of reliability which can be justified. Specifically the following areas of needed research are recognized.

(1) Continued work is needed to determine variability of pavement system design inputs and their individual effects.

(2) Better reliability measures are needed and these must also be calibrated to the actual pavement condition being considered.

(3) The level of reliability needed for various conditions and classes of pavement service must be determined.

(4) More extensive efforts are needed to collect relevant feedback data in order to actually verify reliability which exists for a wide variety of pavement conditions under actual field conditions.

(5) Inherent in this process and particularly in item 4 is the determination of realistic lack-of-fit variance for systems models and the improvement of existing pavement systems methods to account for these errors.

REFERENCES

1. ASCE Committee on Structural Design of Roadways, "Problems of Designing Roadway Structures," Transportation Engineering Journal, Vol 95, No. TE2, Proceedings of the Americal Society of Civil Engineers, May 1969.

2. "The AASHO Road Test, Report 5, Pavement Research," Special Report 61E, Highway Research Board, 1962.

3. Carey, W. N., Jr., and P. E. Irick, "The Pavement Serviceability - Performance Concept," Highway Research Bulletin 250, 1960.

4. Dommasch, D. O., and C. W. Laudeman, Principles Underlying Systems Engineering, Pitman Publishing Corporation, New York, 1962.

5. Ellis, D. O., and F. J. Ludwig, Systems Philosophy, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1962.

6. Haas, R. C. G., and B. G. Hutchinson, "A Management System for Highway Pavements," Paper Prepared for Presentation to Australian Road Research Board, September 1970.

7. Hudson, W. R., F. N. Finn, B. F. McCullough, K. Nair, and B. A. Vallerga, "Systems Approach to Pavement Design," Interim Report, NCHRP Project 1-10, Materials Research and Development, Inc., March 1968.

8. Hudson, W. R., B. F. McCullough, and Fred N. Finn, "Factors Affecting Performance of Pavement Systems," Transportation Engineering Journal, Vol 95, No. TE3, American Society of Civil Engineers, August 1969.

9. Hudson, W. R., B. F. McCullough, F. H. Scrivner, and J. L. Brown, "A Systems Approach Applied to Pavement Design and Research," Research Report 123-1, Texas Highway Department Cooperative Research Program, March 1970.

10. Hutchinson, B. G., and R. C. G. Haas, "A Systems Analysis of the Highway Pavement Design Process," HRB, Highway Research Record No. 239, 1968.

11. Irick, P. E., and W. R. Hudson, "Guidelines for Satellite Studies of Pavement Performance," NCHRP Report 2A, Highway Research Board, 1964.

12. Roberts, Freddy L., and W. Ronald Hudson, "Pavement Serviceability Equations Using the Surface Dynamics Profilometer," Research Report 73-3 Center for Highway Research, The University of Texas at Austin, April 1970.

13. Scrivner, F. H., W. F. McFarland, and G. R. Carey, "A Systems Approach to the Flexible Pavement Design Problem," Research Report 32-11, Texas Transportation Institute, 1968.

14. Wilkins, E. B., "Outline of a Proposed Management System for C.G.R.A.'s Pavement Design and Evaluation Committee," Proceedings, Canadian Good Roads Association, 1968.

15. Asphalt Pavements for Airports, The Asphalt Institute, June 1963, Manual Series No. 11 (MS-11).

16. Hudson, W. R., and Thomas W. Kennedy, "The Airfield Pavement System and its Parameters," A Report submitted to the Project Systems Branch, EDD, Department of the Army, Construction Engineering Research Laboratory, Champaign, Illinois, July 1970.

17. Structural Design of Asphalt Concrete Pavement Systems, HRB SR126, Highway Research Board, 1971.

18. Petersen, Dale, A Report on the Utah Pavement Evaluation System, Utah Highway Dept., 1968-70.

19. Kher, Ramesh K., W. R. Hudson, and B. F. McCullough, "A Systems Analysis of Rigid Pavement Design," Report No. 123-5. Describes the development of a working systems model for the design of rigid pavements.

20. Hudson, W. R. and B. F. McCullough, "Systems Formulation: Development of SAMP - An Operational Pavement Design System, "National Cooperative Highway Research Program Report 139.

21. Nair, Keshavan and C-Y Chang, "Translating AASHO Road Test Findings Basic Properties of Pavement Components: Materials Characterization," NCHRP Project Nos. 1-10 and 1-10/1. National Cooperative Highway Research Program Report 140.

22. Haas, R. C. G., "Developing a Pavement Feedback Data System," Report No. 123-4. Describes the initial planning and development of a pavement feedback data system.

23. Strom, Oren G., W. Ronald Hudson, and James L. Brown, "A Pavement Feedback Data System," Report No. 123-12. Defines a data system to acquire, stores, and analyze performance feedback data from in-service flexible pavements.

24. Haas, Ralph, "Surface Evaluation of Pavements: State-of-the-Art," a paper presented for presentation at the Highway Research Board Workshop on Pavement Rehabilitation, San Francisco, California, September 1973.

25. Hudson, W. R. and F. N. Finn, " A General Framework for Pavement Rehabilitation," a paper presented at the San Francisco Workshop on Pavement Rehabilitation sponsored by FHWA, HRB, September 19-22, 1973.

26. Westgaard, "Analysis of Stresses in Concrete Pavements due to Variation in Temperature" Proceedings, Vol. 5, Highway Research Board, 1956.

27. The WASHO Road Test, Part 1: Design, Construction, and Procedures, Highway Research Board Special Report 18, 1954.

28. The AASHO Road Test Report No. 2, Materials and Construction, National Academy of Sciences, National Research Council, Publication 951, Highway Research Board Special Report 61-B, 1962, table 46, p. 82.

29. A Study in the General Field of Quality Control Engineering, State Road Department of Florida in cooperation with the Bureau of Public Roads, by Warden Engineers, Inc., Raleigh, N.C., September 1965.

30. Determination of Statistical Parameters for Highway Construction, Research Project No. 18, State Road Commission of West Virginia, by Materials Research and Development Inc., Miller Warden Associates Div., Raleigh, N.C., July 1966.

31. Procedures for Using Statistical Methods for Process Control and Acceptance of Bituminous Mixtures, South Carolina State Highway Department, By Paguette-Mills, Consulting Engineers, Atlanta, Ga., February 1966.

32. McMahon, T. F., and W. S. Halstead, Quality Assurance in Highway Construction, Public Roads, Vol. 35, Nos. 6011, 1965, W. S. Bureau of Public Roads.

33. Bituminous Pavement Construction, by M. J. Kilpatrick, and R. G. McQuate, Bureau of Public Roads, June 1967, pp. 19-22, and Figure 16, p. 23.

34. McMahon, T. F., Quality Assurance for Highway Construction in the United States of America, Proceedings, 5th Conf. of the ARRB Vol. 5, Part 5, 1970.

35. Treybig, Harvey J., B. Frank McCullough, and W. Ronald Hudson, "Design Manual for Continuously Reinforced Concrete Overlay Pavements," Vol. II, CRC Airfield Pavement, AFWL Contract No. F 29601-72-0057, Austin Research Engineers Inc., December 1973.

36. Treybig, Harvey J., B. Frank McCullough, and W. Ronald Hudson, "Design Manual for Continuously Reinforced Concrete Pavement," Vol. III, CRC Airfield Pavement, AFWL Contract No. F29601-72-0057, Austin Research Engineers Inc., December 1973.

37. "Preliminary Pavement Recommendations for the Dallas-Fort Worth Regional Airport," A report presented to Forrest and Cotton, and Carter and Burgess Associate Engineers, Dallas, Texas, Austin Research Engineers Inc., March 1971.

38. Treybig, Harvey J., B. Frank McCullough, and W. Ronald Hudson, "Tests on Existing Pavements and Synthesis of Design Methods," Vol. I, CRC Airfield Pavement, AFWL Contract No. F29601-72-0057, Austin Research Engineers Inc., December 1973.

39. Lemer, A. C., and F. Moavenzadeh, "Reliability of Highway Pavements," A paper prepared for presentation at the 50th Annual Meeting of the Highway Research Board, Department of Civil Engineering, Massachusetts Institute of Technology, 1971.

40. Soussou, J. E., F. Moavenzadeh, and H. K. Findakly, "Synthesis for Rational Design of Flexible Pavements - Part II," FHWA Contract No. FH 11-776, MIT, January 1973.

41. Darter, Michael I., and W. R. Hudson, Probabilistic Design Concepts Applied to Flexible Pavement System Design, Research Report 123-18, published by the Center for Highway Research, University of Texas at Austin.

42. Finn, F. N., "Observation of the Stress in Full Scale Pavements," Special Report 126, Highway Research Board, 1971, pages 86-90.

43. Moore, R. K. and T. W. Kennedy, "Tensile Behavior of Stabilized Subbase Materials Under Repetitive Loadings," Research Report No. 98-12, Center for Highway Research, The University of Texas at Austin, 1971.

44. Kher, Ramesh K., and M. I. Darter, Probabilistic Concepts and their Application in AASHO Interim Guide for Design of Rigid Pavements, Highway Research Board, 1973, Washington, D.C.

45. Holsen, Malvin, and W. R. Hudson, Stochastic Study of Design Parameters and Lack-of-fit of Performance Model in the Texas Flexible Pavement Design System, RR 123-23, Center for Highway Research, 1974.

46. Steitle, David Charles, "Development of Criteria for Airport Runway Roughness Evaluation," Thesis, The University of Texas at Austin, December 1972.

47. Kennedy, T. W., W. Ronald Hudson, B. F. McCullough, "Variability of Material Properties forAirport Pavement Systems," by ARE Inc., Engineering Consultants, Austin, Texas, 1974.

48. McCullough, B. Frank, Adnan Abou-Ayyash, W. Ronald Hudson, and Jack P. Randall, "Design of Continuously Reinforced Concrete Pavements for Highways," Research Project NCHRP 1-15, Center for Highway Research, The University of Texas at Austin, August 1974. (Not Approved for Publication)

49. Darter, Michael I., W. Ronald Hudson, and Ralph C. G. Haas, "Selection of Optimal Pavement Designs Considering Reliability, Performance, and Costs." A paper prepared for presentation at the 53rd Annual Meeting of the Highway Research Board, Center for Highway Research, The University of Texas at Austin, January 1974. (Not for Publication)

50. Lytton, R. L. and W. F. McFarland, "Systems Approach to Pavement Design - Implementation Phase," Final Report 1-10A-NCHRP, March 1974.

APPENDIX A. DEFINITIONS OF TERMS

(1) Performance is a measure of the accumulated service provided by a facility, i.e., the adequacy with which a pavement fulfills its purpose. Performance is often specified with a performance index as suggested by Carey and Irick. As such, it is a direct function of the present serviceability history of the pavement.

(2) Serviceability is the ability of a specific section of pavement to serve traffic in its existing condition. (Note that the definition applies to the present (existing) condition - that is, on the date of rating - not to be assumed condition the next day or at any future or past date.)

(3) Behavior is the immediate reaction or response of a pavement to load, environment, and other inputs. Such response is usually a function of the mechanical state, i.e., the stress, strain, or deflection, which occurs in response to the input.

(4) Distress is the visible consequences of various mechanisms of distress which usually lead to a reduction in serviceability.

(5) A system is something which accomplishes an operational process; that is, something is operated on in some way to produce something. That which is operated on is usually input; that which is produced is called output, and the operating entity is called the system. The system is a device, procedure, or scheme which behaves according to some description, its function being to operate on information and/or energy and/or matter in a time reference to yield information and/or energy and/or matter and/or service (Ellis and Ludwig).

(6) Systems failure may be expressed as a condition where the total combined distress in the system response has exceeded an acceptable level based on the decision criteria as when the serviceability level drops below an acceptable level.

(7) Reliability is the probability that the pavement system will perform its intended function over its design life (or time) and under the conditions (or environment) encountered during operation. The four basic elements involved in this concept of pavement system reliability are probability, performance, time, and environment.